Technical Report Documentation Page

1. REPORT No.
FHWA/CA/TL-80/13

2. GOVERNMENT ACCESSION No.

3. RECIPIENT'S CATALOG No.

4. TITLE AND SUBTITLE
In Situ Measurement of Soil Properties

5. REPORT DATE
May 1980

6. PERFORMING ORGANIZATION

7. AUTHOR(S)
S.B.P. John

8. PERFORMING ORGANIZATION REPORT No.
19305-632136

9. PERFORMING ORGANIZATION NAME AND ADDRESS
Office of Transportation Laboratory
California Department of Transportation
Sacramento, California 95819

10. WORK UNIT No.

11. CONTRACT OR GRANT No.

12. SPONSORING AGENCY NAME AND ADDRESS
California Department of Transportation
Sacramento, California 95807

13. TYPE OF REPORT & PERIOD COVERED
Final

14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES
This research was performed under FHWA, HPR Study D-2-20.

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This report presents evaluation of five in situ testing devices. The five devices are Iowa Borehole Shear Device, Vane Shear Device, Standard Penetrometer, Dutch Cone Penetrometer and Cambridge Self Boring Pressuremeter. These devices have been field tested in three Northern California sites, namely: Downtown Sacramento, Rio Vista, and Mare Island. The in situ tests were conducted in silty sand, silty clay, San Francisco bay mud (recent), peat, and peaty clay. Maximum depth tested was 50 feet (~16m). Laboratory tests were conducted on undisturbed samples. The various soil parameters derived from laboratory tests have been compared with those from the field tests. The effects of soil disturbance are discussed. A separate report appendix intend for limited distribution (upon request) contains information on modification of probes, adaptation of drill rigs, calibration, testing procedures, boring profiles, and a bibliography.

17. KEYWORDS
Soil Mechanics, In situ tests, soil properties, Iowa Shear Device, Vane Shear, Standard Penetrometer, Dutch Cone, Cambridge Self Boring Pressuremeter

18. No. OF PAGES:
181

19. DRI WEBSITE LINK
http://www.dot.ca.gov/hq/research/researchreports/1978-1980/80-13.pdf

20. FILE NAME
80-13.pdf

TECHNICAL REPORT STANDARD TITLE PAGE

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17. KEY WORDS	18. DISTRIBUTION STATEMENT
Soil Mechanics, In situ tests, soil properties, Iowa Shear Device, Vane Shear, Standard Penetrometer, Dutch Cone, Cambridge Self Boring Pressuremeter.	No restrictions. This document is available to the public through the National Technical Information Service, Springfield VA 22161.

19. SECURITY CLASSIF. (OF THIS REPORT)	20. SECURITY CLASSIF. (OF THIS PAGE)	21. NO. OF PAGES	22. PRICE
Unclassified	Unclassified	181	

DS-TL-1242 (Rev.6/76)

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF CONSTRUCTION
OFFICE OF TRANSPORTATION LABORATORY

May 1980

TL No. 632136

IN SITU MEASUREMENT OF SOIL PROPERTIES

Study Made by	Soil Mechanics and Pavement Branch
Under the Supervision of	R. A. Forsyth, P.E.
Principal Investigators	R. H. Prysock, P.E. S. B. P. John, P.E.
Research Conducted and Report Prepared by	S. B. P. John, P.E.
Assisted by	J. G. Macfarlane L. R. Leech A. Y. Lee, P.E. John Campbell

APPROVED BY

NEAL ANDERSEN
Chief, Office of Transportation Laboratory

ACKNOWLEDGEMENTS

The work reported herein is the result of an extensive research project authorized under HPR D-2-20 in Cooperation with the Federal Highway Administration, U.S. Department of Transportation.

Personnel of the former Geotechnical Branch and the present Soil Mechanics and Pavement Branch of the Transportation Laboratory who assisted with this research project include:

Jim Macfarlane, who conducted the in situ tests with the Dutch Cone Penetrometer, Cambridge Self Boring Pressuremeter, Iowa Device, and Vane Shear Device; designed the console for the Cambridge Probe; and analyzed data developed during the investigation.

Ray Leech, who conducted the in situ tests with the Vane Shear Device, Standard Penetrometer, and Iowa Device.

Dave Mauldin, who designed the worktable for the Iowa Device and the cutter drive unit for the Cambridge Probe.

George Oki, Dick Carello, and Leonard Nordmann, who designed and fabricated the mounting bracket for the Dutch Cone Device.

John Nunes, who designed the cutting shoe and the cutter for the Cambridge Probe.

Albert Lee and Susan Gallardo, who analyzed the data and plotted the correlations and comparisons.

John Campbell, who edited the final draft; Marion Ivester, Elmer Wigginton, Bill Taylor, and Eddie Fong, who prepared the figures for this report. Typing was by Darla Bailey.

CONVERSION FACTORS

English to Metric System (SI) of Measurement

Quanity	English unit	Multiply by	To get metric equivalent
Length	inches (in)or(")	25.40 .02540	millimetres (mm) metres (m)
	feet (ft)or(')	.3048	metres (m)
	miles (mi)	1.609	kilometres (km)
Area	square inches (in^2) square feet (ft^2) acres	6.432×10^{-4} .09290 .4047	square metres (m^2) square metres (m^2) hectares (ha)
Volume	gallons (gal) cubic feet (ft^3) cubic yards (yd^3)	3.785 .02832 .7646	litres (l) cubic metres (m^3) cubic metres (m^3)
Volume/Time			
(Flow)	cubic feet per second (ft^3/s)	28.317	litres per second (l/s)
	gallons per minute (gal/min)	.06309	litres per second (l/s)
Mass	pounds (lb)	.4536	kilograms (kg)
Velocity	miles per hour(mph) feet per second(fps)	.4470 .3048	metres per second (m/s) metres per second (m/s)
Acceleration	feet per second squared (ft/s^2)	.3048	metres per second squared (m/s^2)
	acceleration due to force of gravity(G)	9.807	metres per second squared (m/s^2)
Weight Density	pounds per cubic (lb/ft^3)	16.02	kilograms per cubic metre (kg/m^2)
Force	pounds (lbs) kips (1000 lbs)	4.448 4448	newtons(N) newtons (N)
Thermal Energy	British thermal unit (BTU)	1055	joules (J)
Mechanical Energy	foot-pounds(ft-lb) foot-kips (ft-k)	1.356 1356	joules (J) joules (J)
Bending Moment or Torque	inch-pounds(ft-lbs) foot-pounds(ft-lbs)	.1130 1.356	newton-metres (Nm) newton-metres (Nm)
Pressure	pounds per square inch (psi) pounds per square foot (psf)	6895 47.88	pascals (Pa) pascals (Pa)
Stress Intensity	kips per square inch square root inch (ksi $\sqrt{in}$)	1.0988	mega pascals $\sqrt{metre}$ (MPa $\sqrt{m}$)
	pounds per square inch square root inch (psi $\sqrt{in}$)	1.0988	kilo pascals $\sqrt{metre}$ (KPa $\sqrt{m}$)
Plane Angle	degrees (°)	0.0175	radians (rad)
Temperature	degrees fahrenheit (F)	$\frac{tF - 32}{1.8} = tC$	degrees celsius (°C)

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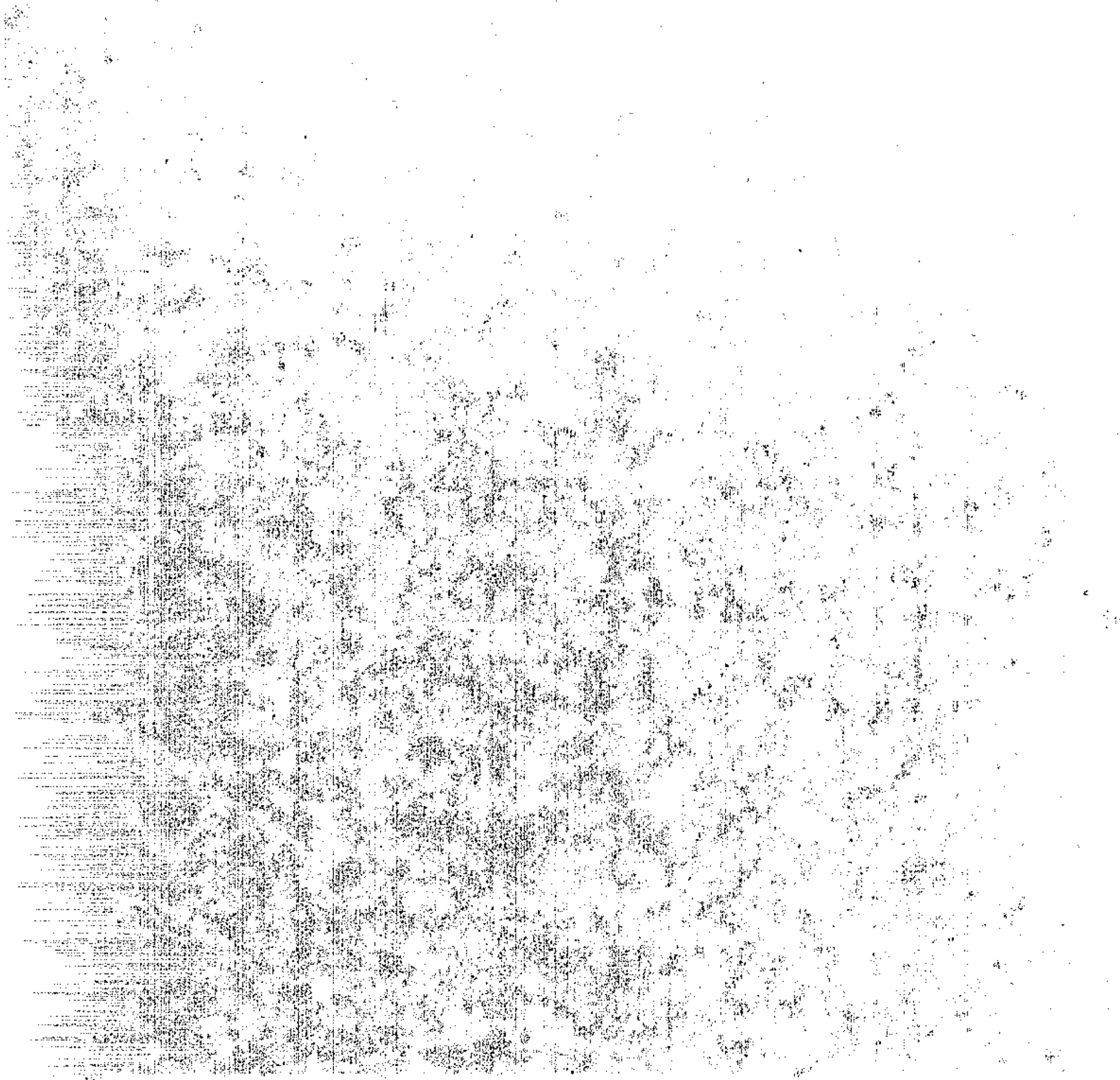

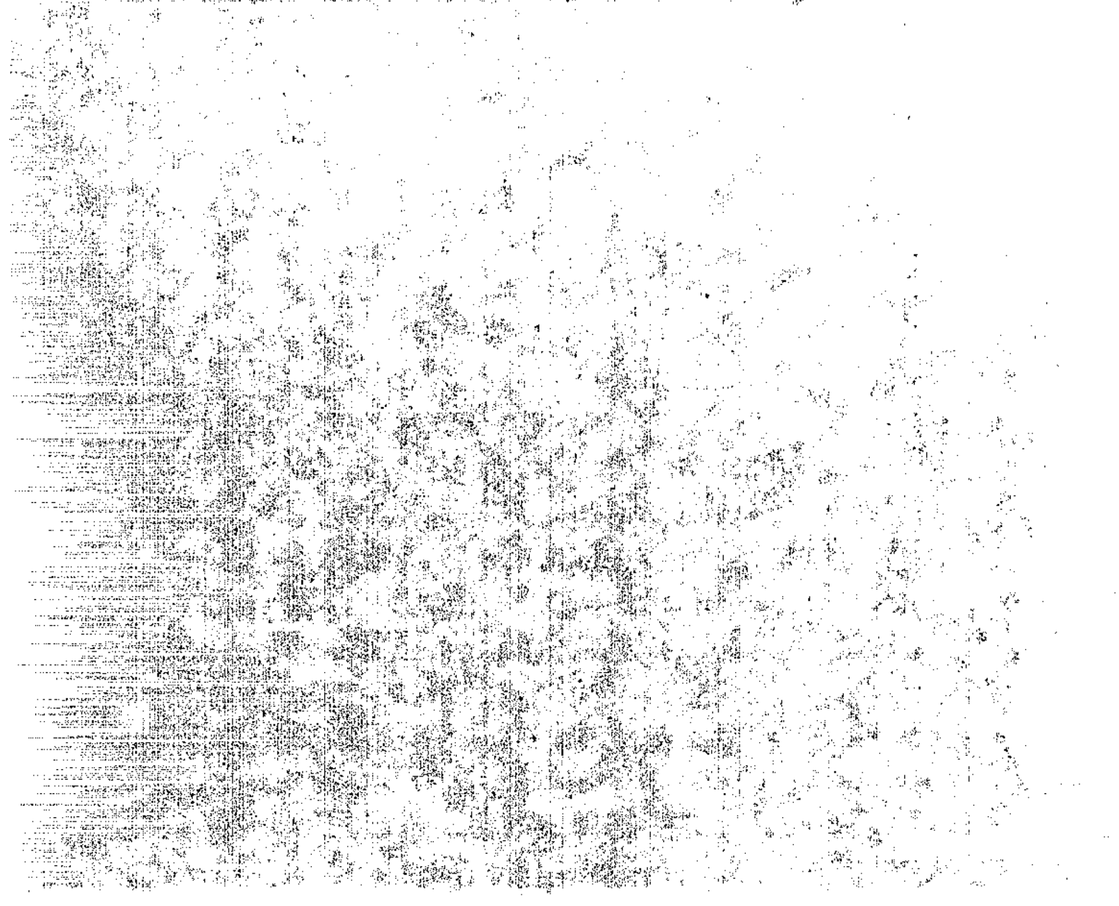

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CHAPTER I. INTRODUCTION

In recent times it has been found increasingly necessary to found major structures and facilities on marginal foundation materials due to a declining number of ideal building sites. Only the careful application of the principles of soil mechanics can assure sound foundation design where the soils involved are weak or sensitive to imposed loads. The principal tool of a geotechnical engineer in assessing subsurface conditions is the information provided by an adequate exploration of the area and determination of soil properties through representative sampling and testing. The need for greater reliability in the evaluation of weak soils has resulted in the development of a new family of in situ soil testing equipment. The purpose of this study was a comprehensive evaluation of several of the more promising of these devices.

Soils are rarely homogeneous. Their properties are subject to significant variation vertically and horizontally. These properties can be altered by boring, sampling, and testing procedures; or by handling in the laboratory. As a result, the mechanical properties determined in the laboratory may often differ appreciably from those in situ.

Furthermore, certain soils as found in nature cannot be sampled satisfactorily or prepared properly for laboratory tests regardless of the degree of care exercised. These include soils with a considerable secondary structure such as fissures, joints, slickensides or concretions, and those containing particles of rock or shells of appreciable size. It is, therefore, often desirable to test such soils as they naturally occur.

Although the prime factor favoring the in situ testing concept is improved accuracy, there is a secondary compelling benefit: economics. The high cost of subsurface exploration has, in many cases, inhibited its utilization to the maximum desirable extent. It is probable that in situ equipment in the hands of experienced operators can produce a greater quantity of reliable data at less cost than conventional sampling and testing methods now generally employed.

The in situ devices evaluated as part of this project can be classified by three general types: 1) shear devices, 2) penetration devices, and 3) pressuremeters, as shown by Figure 1. The shear devices evaluated were the Iowa Borehole and the Vane Shear. The penetration devices were the Standard and Dutch Cone penetrometers. The Cambridge pressuremeter was also evaluated as part of this project. Two French self-boring pressuremeters were evaluated recently by this Department, for another federally financed project(1,2).

Objectives

The two main objectives of this research project were:

1. To evaluate the reliability and limitations of the five in situ measuring instruments previously listed.
2. To determine the capacities of the equipment to fulfill the requirements of the Department of Transportation and develop procedures for its employment.

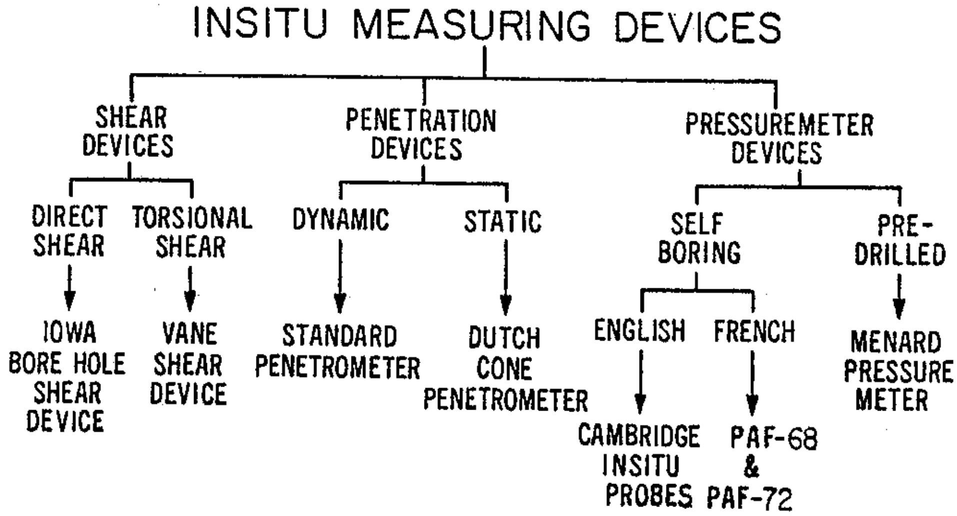


Figure 1 CLASSIFICATION OF IN SITU DEVICES

Scope

The project was initiated in fiscal year 1972-73 by the Geotechnical Branch of the Caltrans Transportation Laboratory. Its scope, which was subsequently expanded, is summarized as follows: 1. Select several sites that contain representative soils suitable for testing in situ probes; 2. Conduct an undisturbed sampling program involving the recovery of both 2- and 3-inch diameter samples; 3. Perform laboratory tests to determine soil properties under laboratory conditions; 4. Adapt and modify the five probes so that they can be used with available Caltrans drilling equipment; 5. Evaluate the five probes in the field; 6. Compare the parameters derived from the field in situ tests with the soil parameters derived from the laboratory testing; 7. Compare the strength parameters obtained from 2- and 3-inch diameter samples in order to establish the effect of this variation in sample size; 8. Develop test procedures for the subject probes.

CHAPTER 2. SITE DESCRIPTIONS AND FIELD TESTING PROGRAM

The field testing program consisted of delineating the various soil layers present at the test sites, collecting representative "undisturbed" samples for laboratory tests, and conducting field tests with the five in situ probes.

From a group of more than ten sites, three were chosen for the project. Their locations are shown in Figure 2.

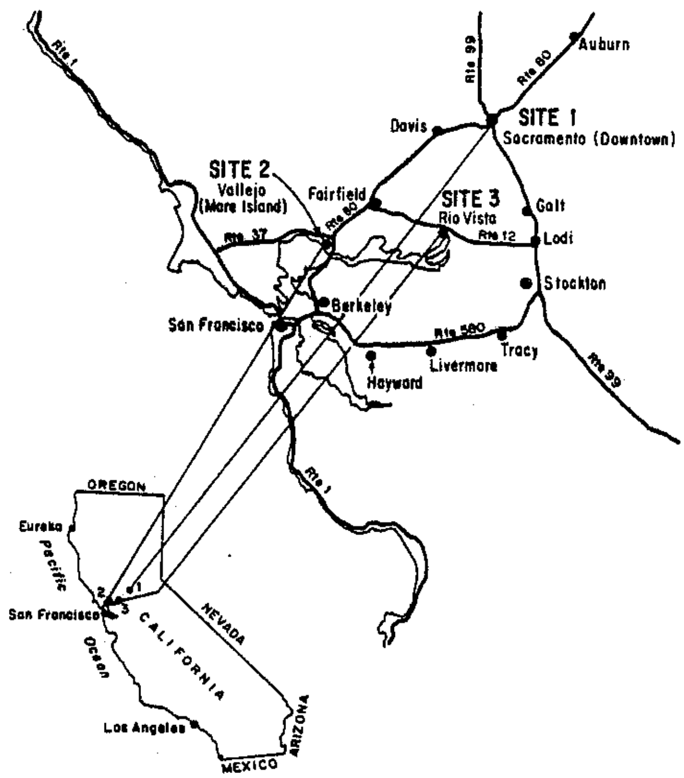


Figure 2 MAP SHOWING LOCATIONS OF SITES 1-3

Site 1 - Downtown Sacramento

The preliminary design studies for Interstate Freeway 80 through the City of Sacramento included foundation exploration for structure approach embankments. The freeway alignment traverses a park located about six city blocks east of the Sacramento River crossing. A natural lake is situated on the south side of this park.

Original foundation exploration in the area of the lake revealed that the very soft silty clay deposit in the area was too deep for stripping. The freeway design in this area was accordingly modified from embankment to structure. Approximately 200 feet of the south end of the lake was dredged to remove several feet of the extremely soft lake bottom deposits. This area was dewatered and backfilled with imported silty sand from local sources. Site 1 is located over this area. Construction was completed in 1962. A generalized soil profile for this area is shown in Figure 3.

Site 2 - Mare Island

Site 2 is located where State Highway Route 37 crosses the Napa River, close to the U.S. Naval Base at Mare Island, California. During exploration preliminary to the construction of a new river-crossing structure, the area foundation soil was determined to be soft, highly plastic clay to depths in excess of 45 feet. This relatively homogeneous, low-strength, highly compressible soil is typical of the soft bay mud deposits found in the San Francisco Bay Area. A generalized soil profile is shown in Figure 4. The test site is located outside the area affected by highway construction.

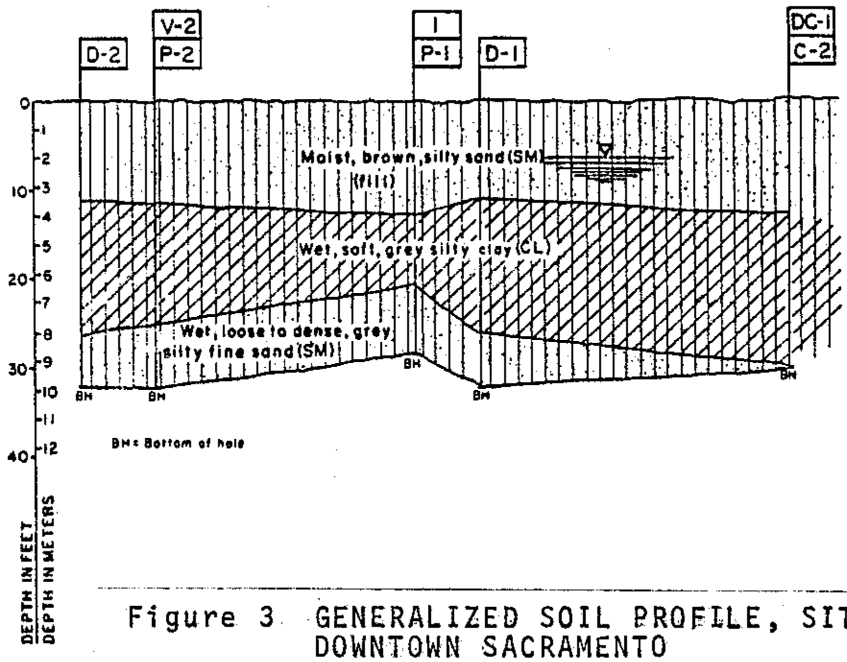


Figure 3 GENERALIZED SOIL PROFILE, SITE 1, DOWNTOWN SACRAMENTO

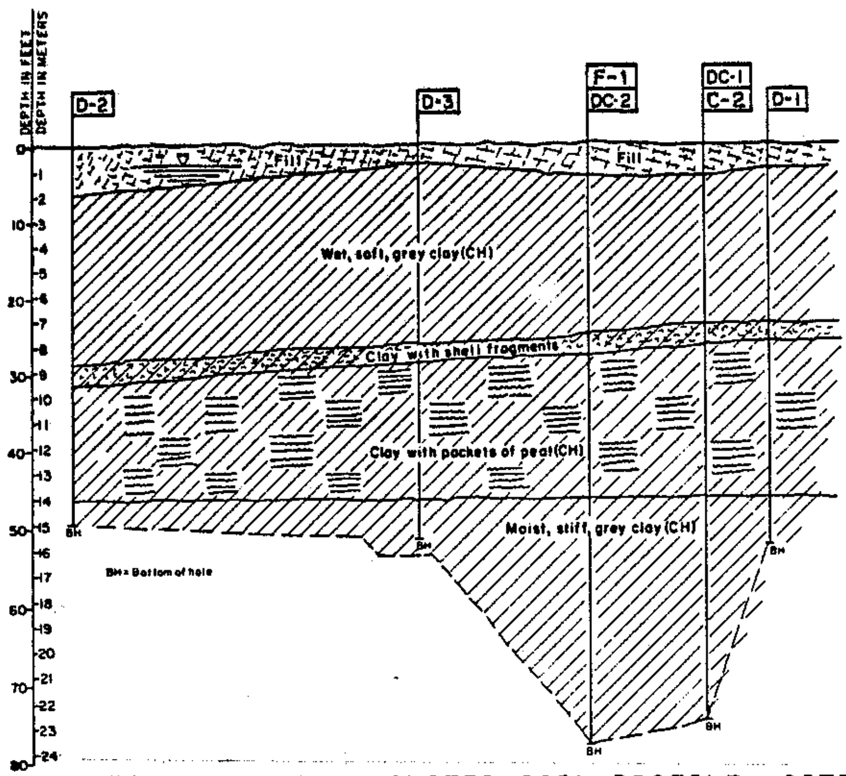


Figure 4 GENERALIZED SOIL PROFILE, SITE 2, MARE ISLAND

Site 3 - Rio Vista

The agricultural lowlands near the Sacramento River at Rio Vista, California, are protected from overflow by levees approximately 20 feet in height along both banks. Paved public roadways are maintained atop these levees. State Highway Route 12, which connects Lodi, on Highway 99, with Rio Vista, is a two-lane undivided road.

An approach embankment for the Route 12 levee/river crossing structure was constructed in 1958. Because of the weak compressible foundation soil to depths of 35 feet, sand drains were installed under the embankment to accelerate the rate of primary settlement. Imported silty sand was placed over the construction area to provide a working table for heavy equipment during construction. Right-of-way acquisition included additional width to allow for pushups resulting from foundation problems or failures, and for possible future widening. A portion of the extra width adjacent to the roadway shoulder in an area unaffected by the fill construction was selected as Site 3. A generalized soil profile is shown in Figure 5.

Layout of Test Holes

The typical boring layout for Sites 1, 2, and 3 is presented in Figure 6. Boreholes D-1, D-2, and D-3 were used for recovering continuous undisturbed samples. Each of the five in situ probes was allotted two holes. Their designation codes are shown in Figure 6, along with the test locations for the two French probes evaluated under a separate project.

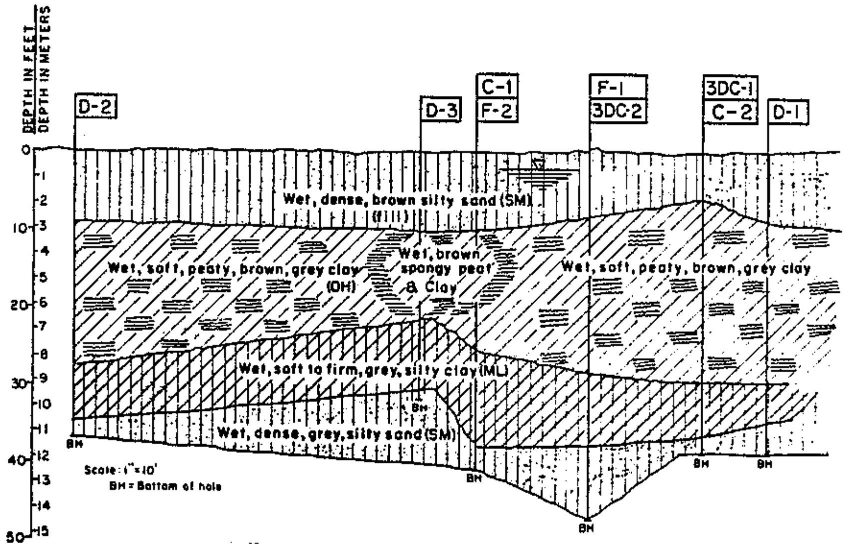
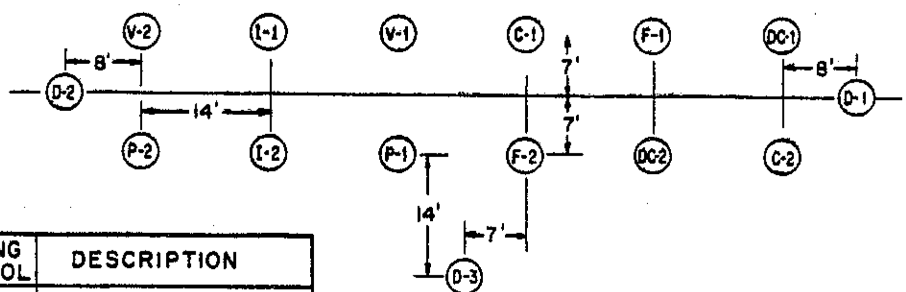


Figure 5 GENERALIZED SOIL PROFILE, SITE 3, RIO VISTA



BORING SYMBOL	DESCRIPTION
D	Drive sample boring
V	Vane shear
P	Standard Penetrometer
C	Cambridge Pressuremeter
F	French Pressuremeters
DC	Dutch Cone
I	Iowa Shear Device

Note: 1 ft. = 30.5 cm

Figure 6 TYPICAL BORING LAYOUT

In Situ Test Program

Soil layers tested in situ were selected based on a review of boring logs and laboratory test data. Two soils at Site 1, and three at Sites 2 and 3, were considered suitable for in situ testing. The maximum depths to which each of the probes were used at the three sites are presented in Table 1.

TABLE 1. MAXIMUM DEPTHS REACHED BY PROBES

Probe	Site 1	Site 2	Site 3
Dutch Cone	12.2	2.3	15.6
Cambridge Probe	8.8	14.6	12.5
Iowa Device	1.6	2.0	1.7
Vane	6.5	12.6	7.5
Standard Penetrometer	9.2	34.7	12.1

Note: 1. Depths are in meters.
2. 1 meter = 3.28 feet.

CHAPTER 3. LABORATORY INVESTIGATION

Laboratory testing of soil samples (2-inch and 3-inch) included routine classification tests, mineral analyses, unconfined triaxial compression tests, and consolidation tests.

Classification Tests

In addition to grain size analyses, Atterberg Limits tests were conducted for the soils from Sites 1, 2, and 3. The results of this latter series are plotted on plasticity charts (Figures 7 through 9). The variation of plastic limits with depth are presented in Figures 10, 11, and 12. The variation of liquid limits with depth are presented in Figures 13, 14, and 15.

Strength Tests

A series of unconfined compressive strength tests was conducted for the undisturbed state and for the after remolding state to evaluate the sensitivity of the clayey soils.

For Site 1, the sensitivity ranged from 1.3 to 3.2 and averaged 1.8. For Site 2, tests gave a range of .7 to 4.7 with an average of 2.0. For Site 3, the range was from .8 to 7.4 with an average of 3.5. These values are plotted in Figure 16.

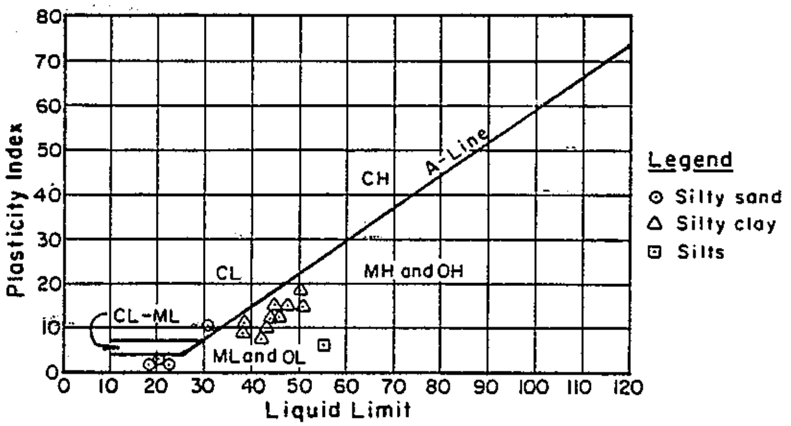


Figure 7 PLASTICITY CHART, SITE 1

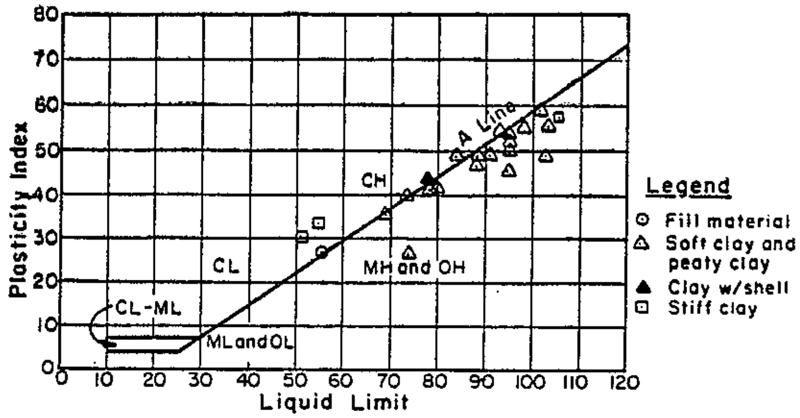


Figure 8 PLASTICITY CHART, SITE 2

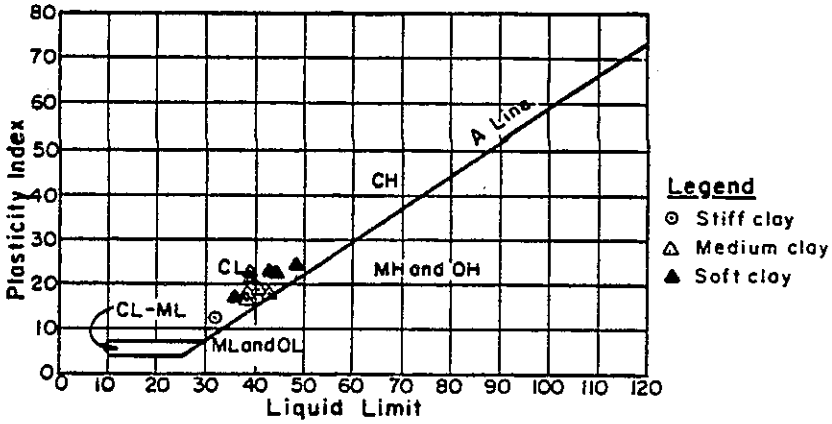


Figure 9 PLASTICITY CHART, SITE 3

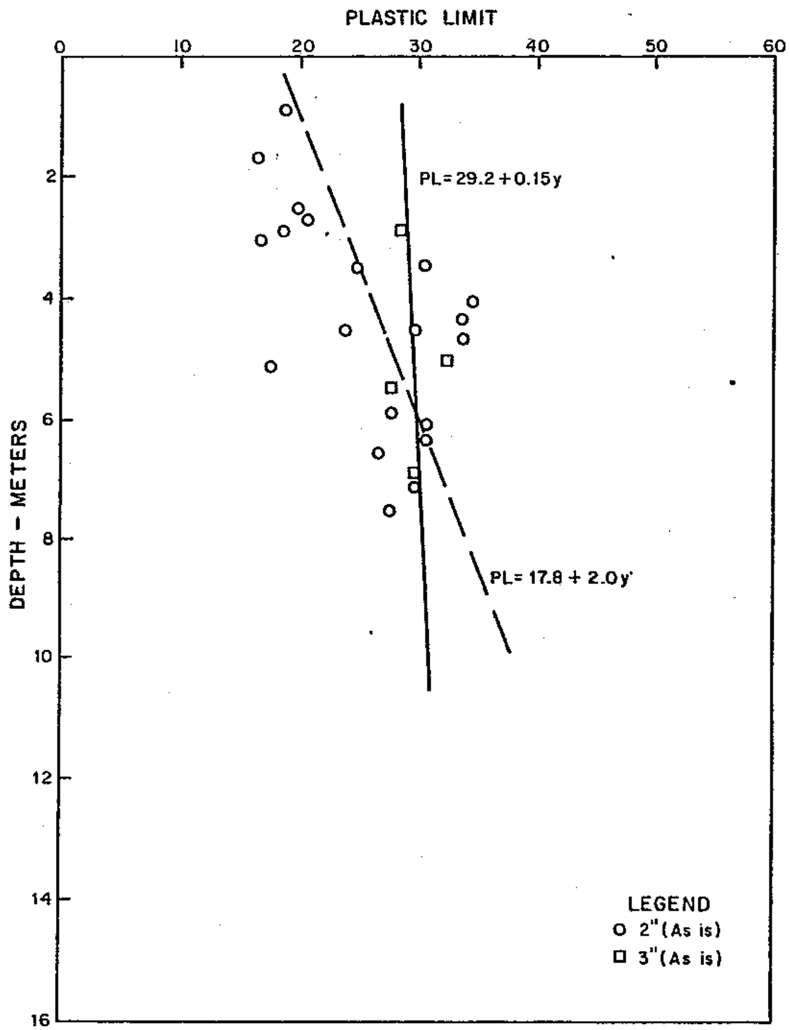


Figure 10 PLASTIC LIMIT STUDY, SITE 1

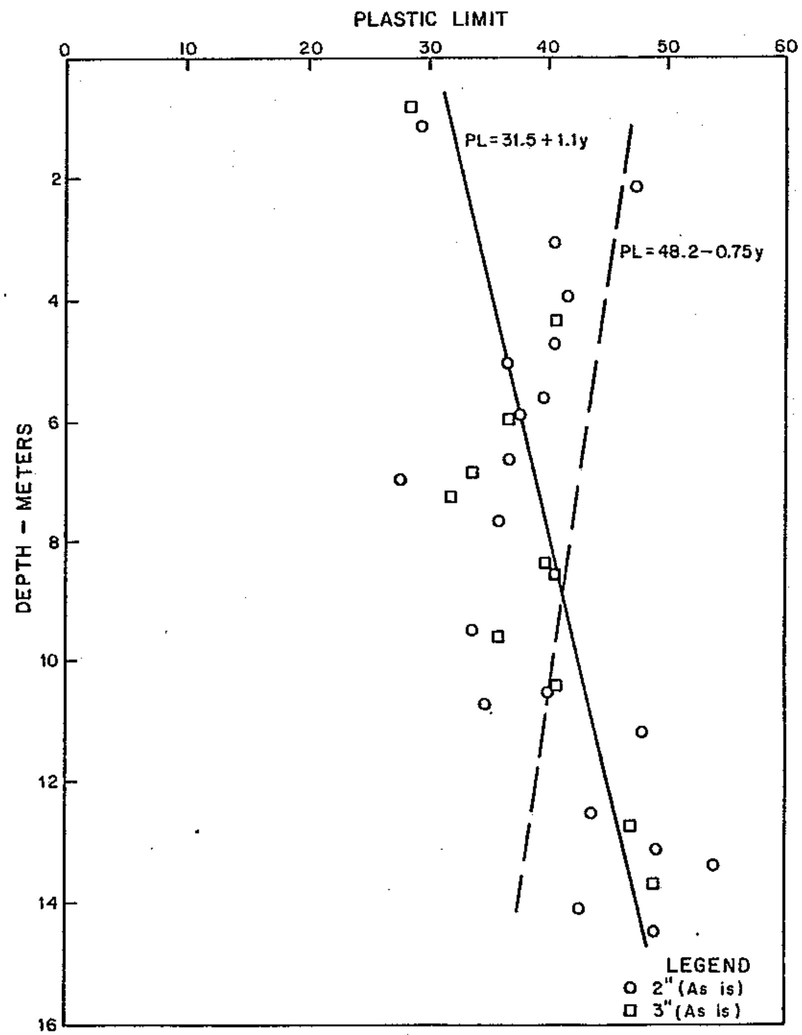


Figure 11 PLASTIC LIMIT STUDY, SITE 2

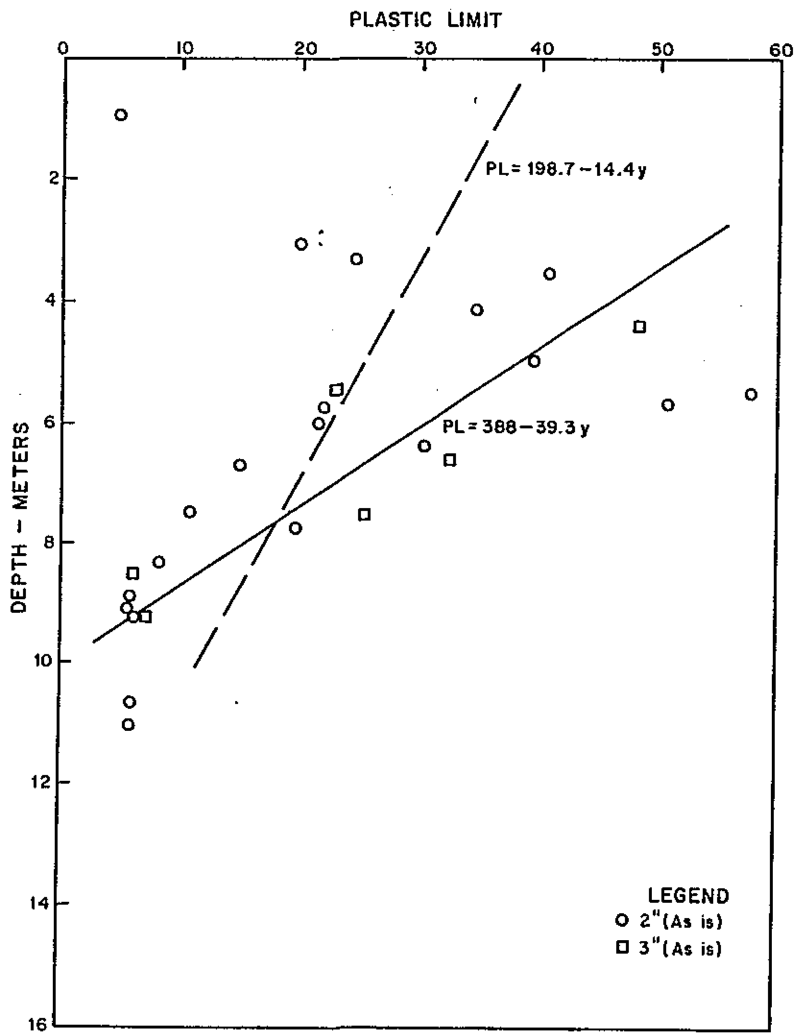


Figure 12 PLASTIC LIMIT STUDY, SITE 3

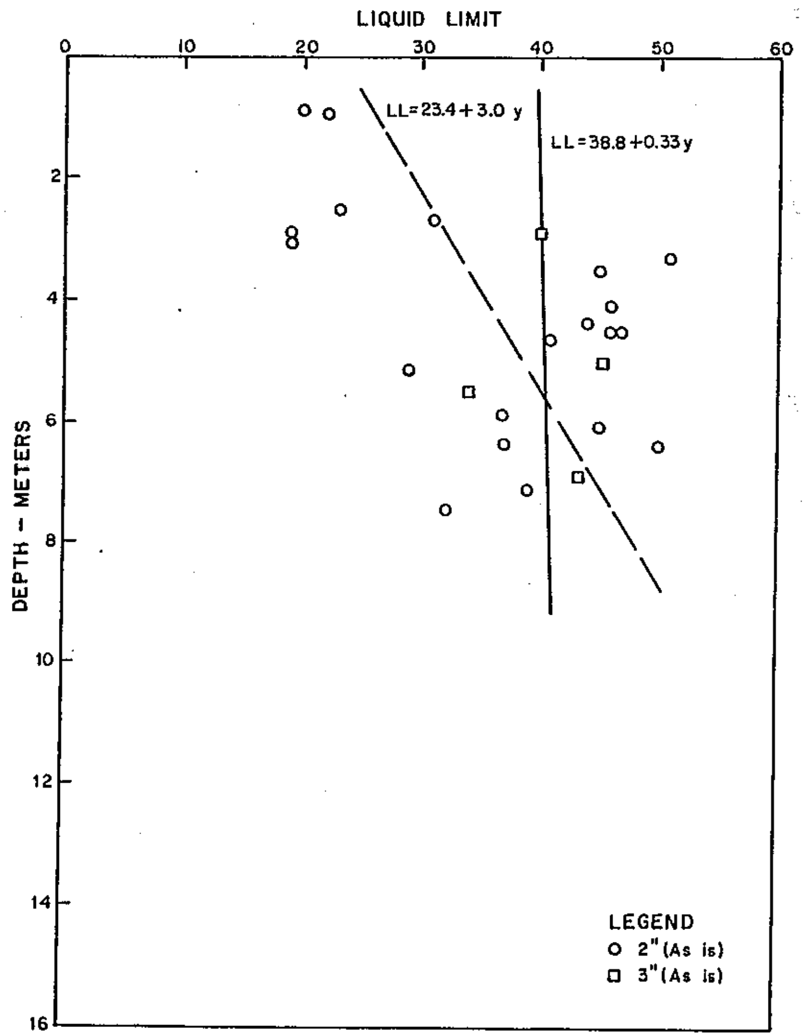


Figure 13 LIQUID LIMIT STUDY, SITE 1

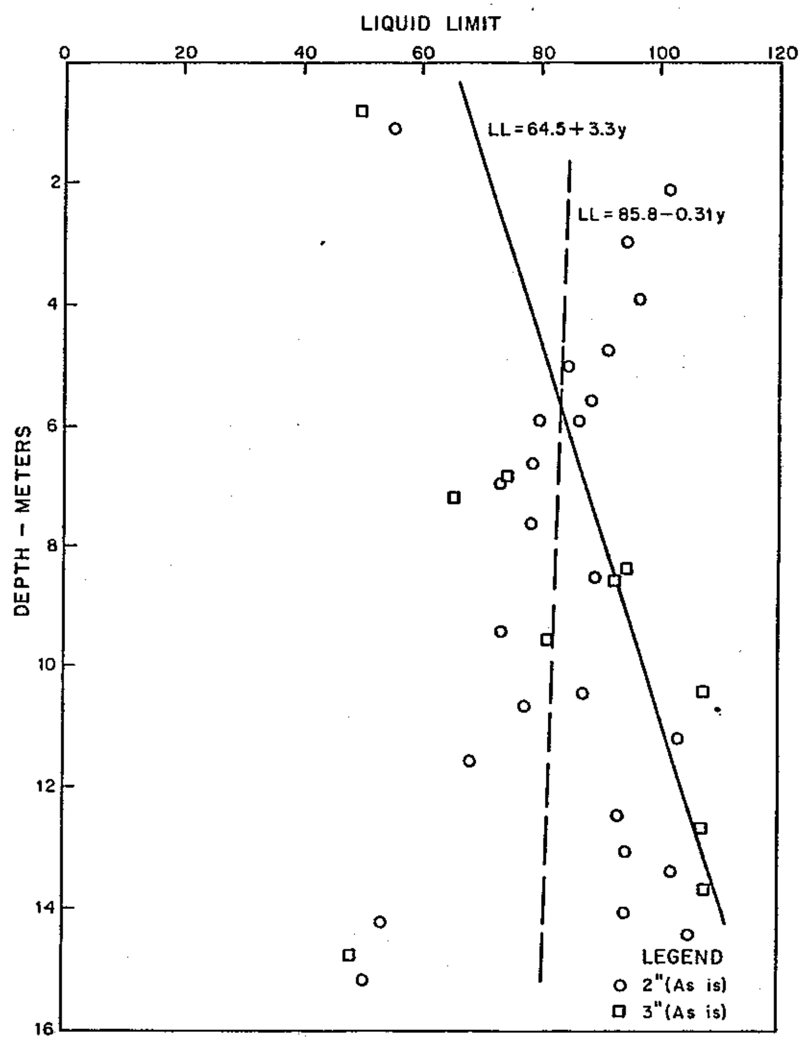


Figure 14 LIQUID LIMIT STUDY, SITE 2

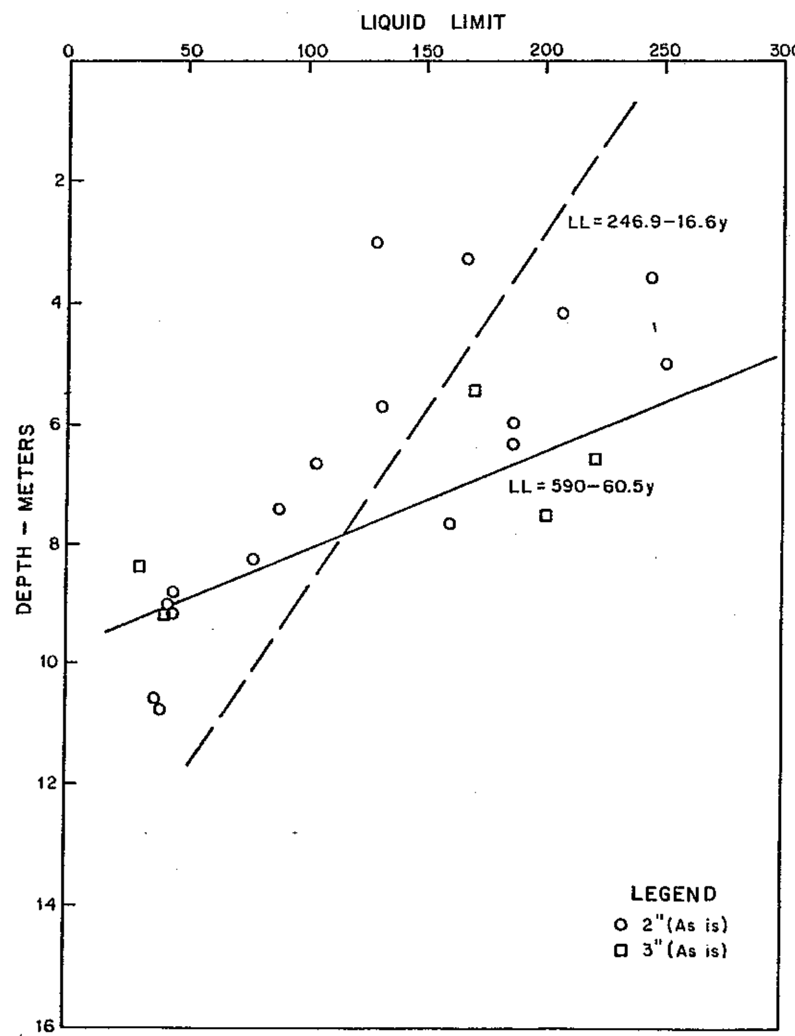


Figure 15 LIQUID LIMIT STUDY, SITE 3

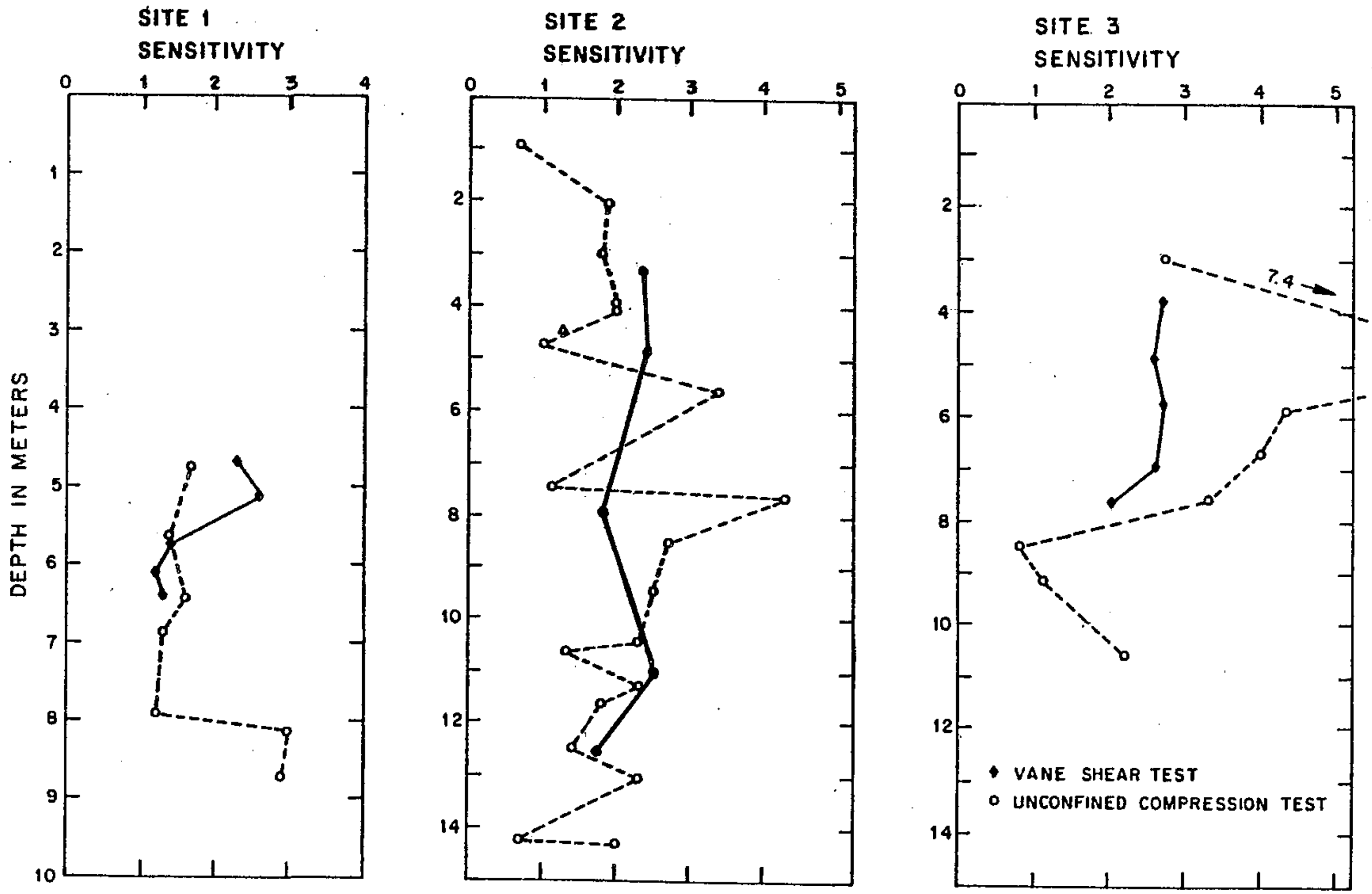


Figure 16 SENSITIVITY STUDY, SITES 1 TO 3

A number of triaxial tests, either unconsolidated-undrained (UU) or consolidated-undrained with pore pressure measurements (CUe), were conducted on samples from the three test sites. A number of unconfined compressive strength tests were also conducted. Shear strength values and other test data are summarized in Figures 17 through 19. Torvane and hand penetrometer tests were also made for comparison purposes. Resulting data are presented in Figure 20.

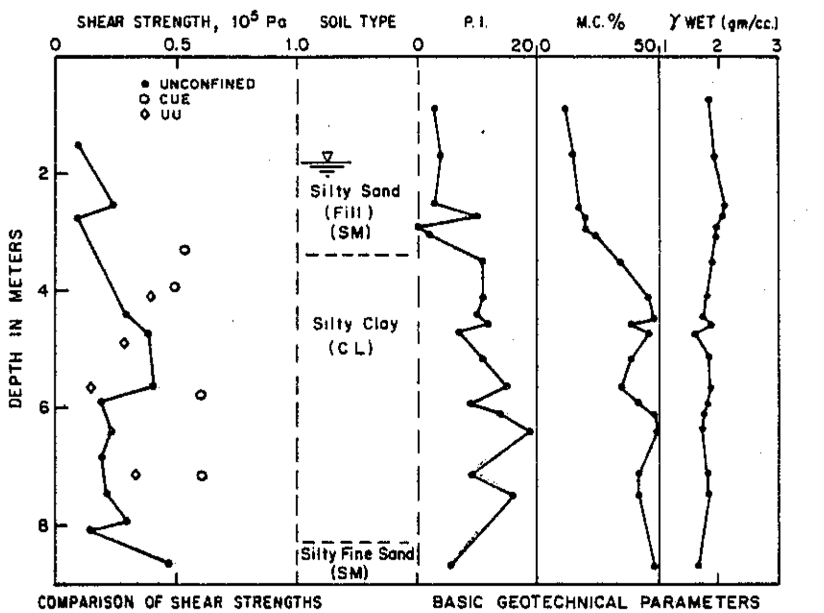


Figure 17 LABORATORY TEST DATA, SITE 1

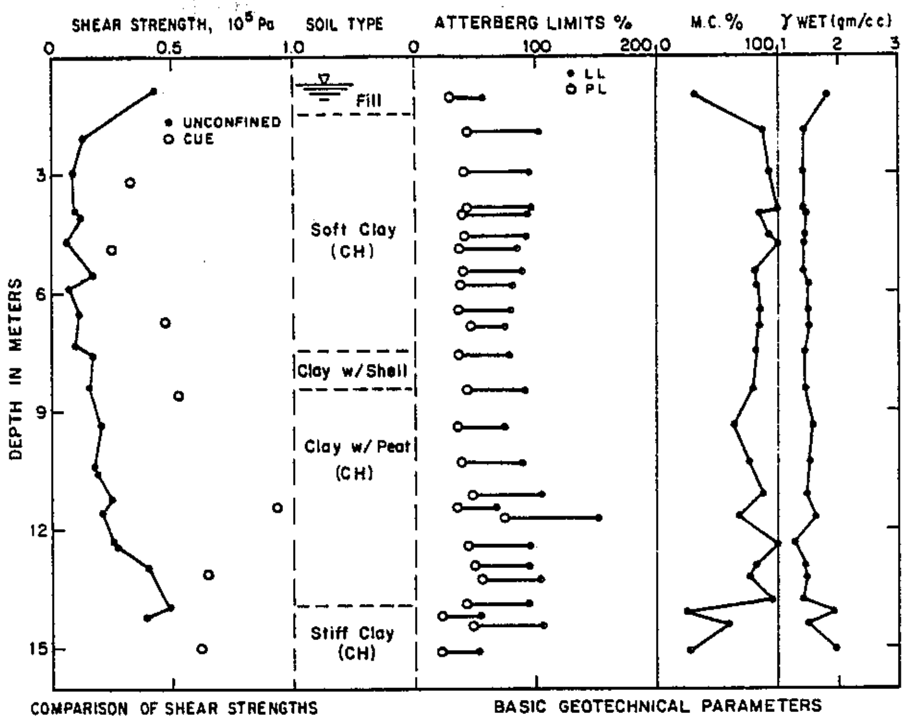


Figure 18 LABORATORY TEST DATA, SITE 2

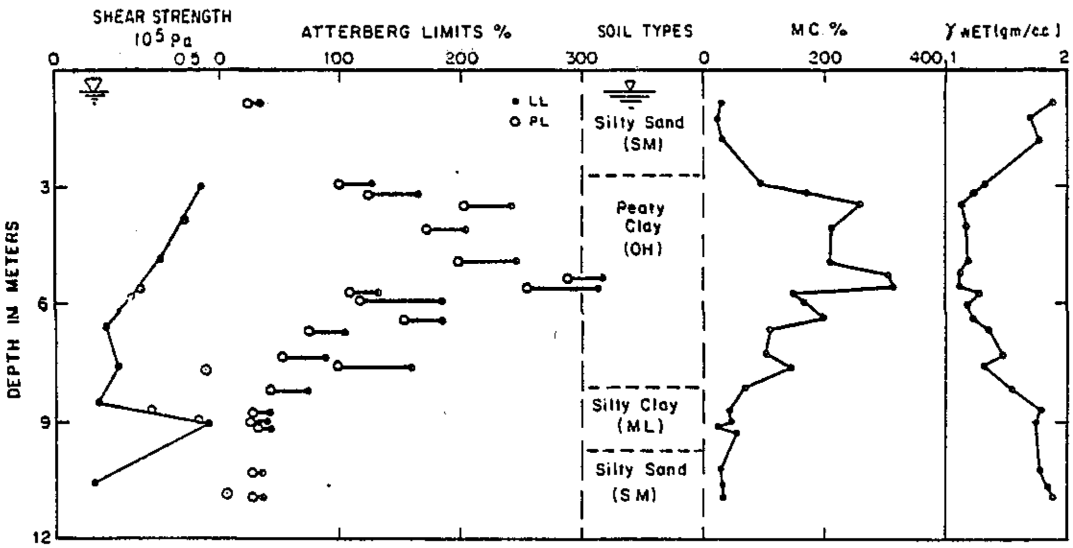


Figure 19 LABORATORY TEST DATA, SITE 3

Note .3048 meter = 1 Foot
1.073×10^5 Pa = 1 tsf

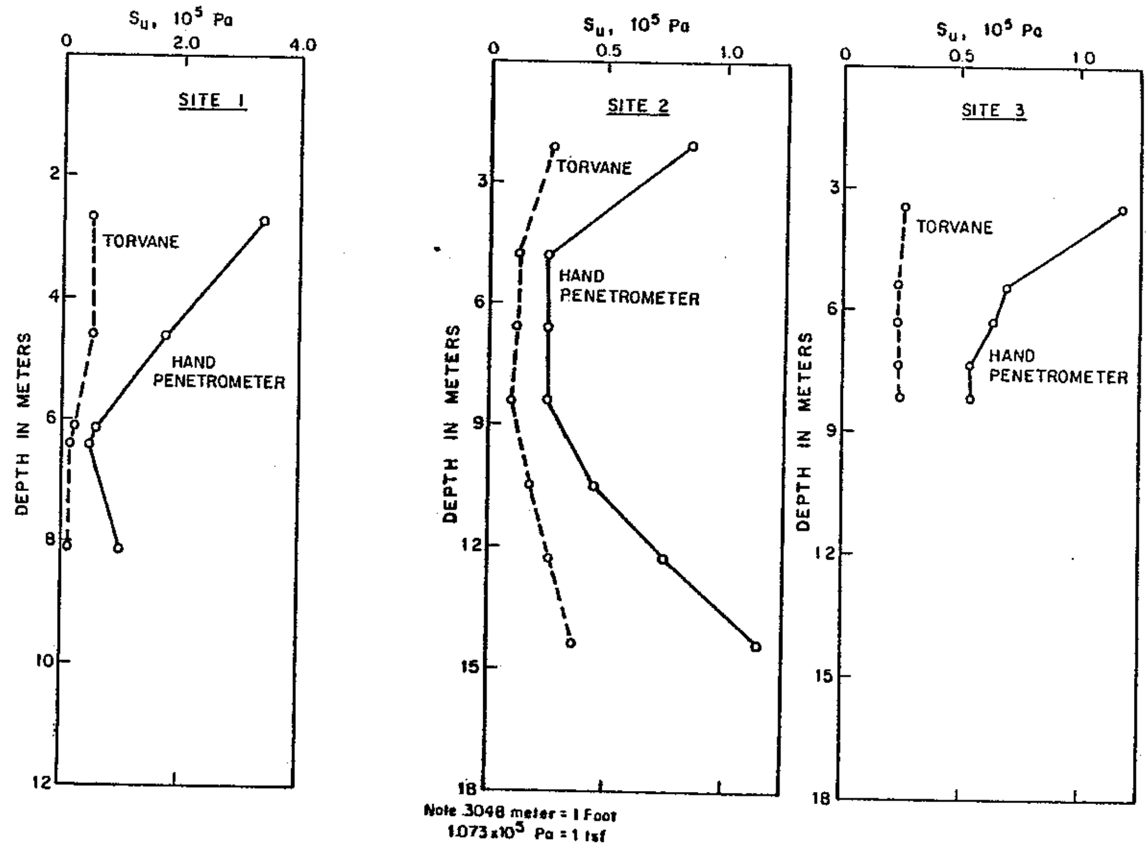


Figure 20 SHEAR STRENGTH PROFILES, SITES 1 TO 3

Consolidation Tests

Consolidation tests were conducted to determine the stress history, as well as the compressibility characteristics at all test sites. The stress histories are presented in Figure 21. The effective overburden pressure (P'_o) and preconsolidation pressure (P'_p) are plotted with respect to depth in Figure 21. The over-consolidation ratio (OCR) varies from 1.0 to 4.2 at Site 1, and from 1.0 to 1.8 at Site 3. Site 2 has normally consolidated soils. The OCR values are plotted in Figure 22.

Clay Mineral Identification

Selected samples from clayey layers were subjected to differential thermal analyses and x-ray diffraction to determine the types and percentages of minerals present. These data are shown in Table 2.

Boring Profiles

Ten boring profiles which summate all the geotechnical information (both subsurface and laboratory test data) are presented as Appendix 7 in a separate report.

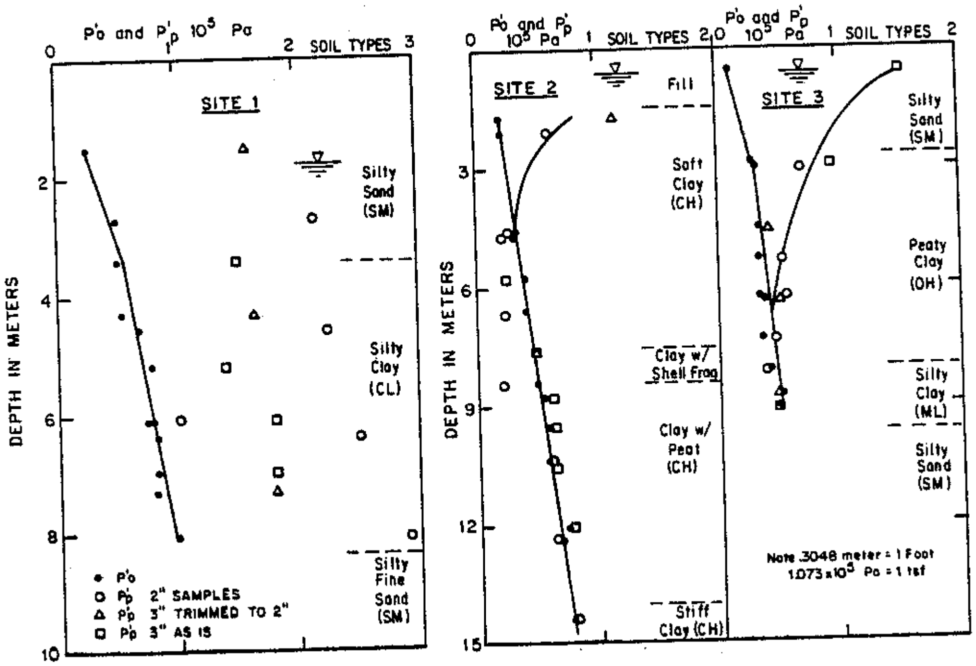


Figure 21 STRESS HISTORY, SITES 1 TO 3

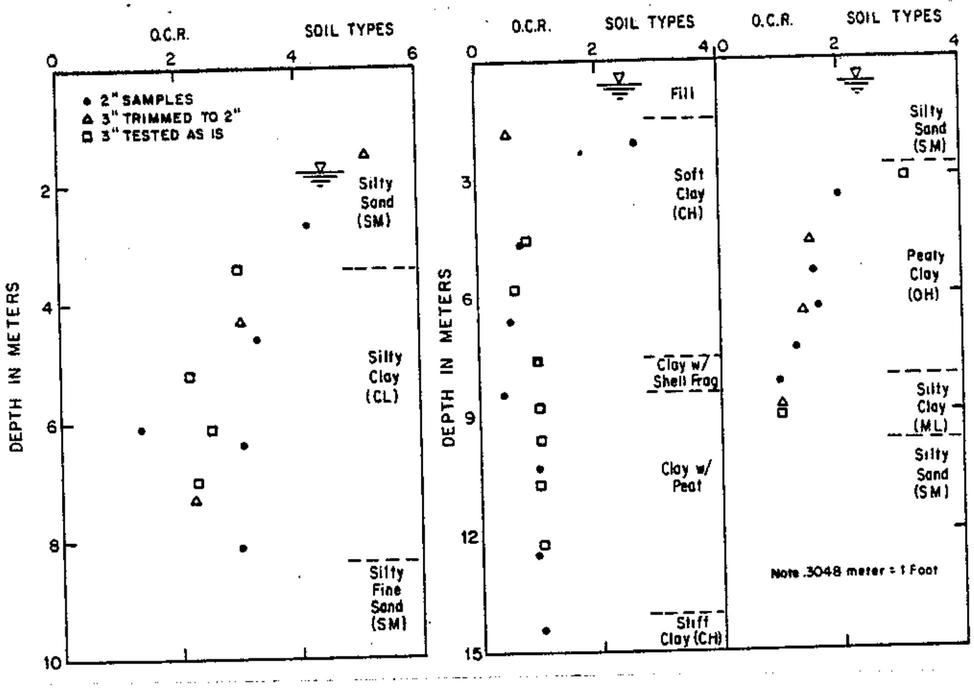


Figure 22 OVERCONSOLIDATION RATIO, SITES 1 TO 3

TABLE 2. X-RAY DIFFRACTION AND DIFFERENTIAL THERMAL ANALYSES

MINERAL / Sample	Minerals Present and Estimated % of Each: Site 1 - Hole D1		Site 2 - Hole D1				Site 3 - Hole DC1		
	1	2	3	4	5	6	7	8	9
Quartz	∿29	20	12-15	15	15	15	15	10	15-20
Feldspar	20-30	25	15-20	∿10	20-25	12-15	10	10	20-25
Chlorite	5-10	5-10	10	5-10	5	>5<10	7.5-10	7.5-10	5-10
Muscovite	5-10	5-10	5-10	5-10		5	5	5	
Iron Oxide	5	<5	<5	5	<5	<5			<5
Iron Sulfide	TR	TR			5		<5	<5	
Organic	TR	<5	∿10	∿5	5	40-45	∿45	∿50	<5
Amphibole	5	5		<5					5
Gibbsite	TR	<5	5	5					
Mica					5				5-10
Talc		<5							
Pyrite						>5			
Calcite					<5				5
Others	5	5		5		5	5	5	5
Montmorillonite, vermiculite, some Mixed Layer Clay	20-25								
Montmorillonite & Mixed Layer Clay, Swelling Type			15						
Montmorillonite, Halloysite, Mixed Layer Clay					30-35				
Vermiculite & Halloysite								>5<10	
Vermiculite & Random Mixed Layer Clay					>5<10				
Halloysite + Kaolinite			15	∿20					
Mixed Layer Clay & Halloysite							7.5		
Random, Mixed Layer Clay, Swelling Type				∿10					20
Halloysite (Probable)		∿20							
DEPTH(METERS)	3.66	5.49	3.96	7.62	10.4	14.0	3.35	5.79	9.45

Note: 1 meter = 3.28 feet.

CHAPTER 4. IOWA BOREHOLE SHEAR DEVICE

Operational Concept

The Iowa Borehole Shear Device was originally conceived to predict skin friction on piles by Dr. Richard Handy(3) at Iowa State University. Initial tests and development were made by N. S. Fox(3) for his Master's thesis at Iowa State. Additional improvements were made by Easton(4) and Andersen(4) in their Master's Degree Programs.

This device permits the rapid determination of in situ shear strength at any point in a predrilled borehole. In essence, the test consists of applying a normal force to shear plates against the sides of the borehole and measuring the shear force required to obtain yielding or failure as the device is pulled vertically. Repetition of this procedure at increasing pressures, or stages, will yield points that can be plotted to produce a Mohr envelope, from which values of cohesion and friction angle can be determined. The maximum pulling force divided by the two contact plate areas gives shear strength; whereas, the expansion force divided by one plate area gives the normal stress.

Description of Device

The Iowa Borehole Shear Device will, henceforth, be referred to as the Borehole Shear Device (BSD). It consists of three basic parts: the shearhead, the pulling device, and the console. The shearhead, shown in Figure 23, consists of two grooved plates, one of which is attached to the piston rod. The double-acting rolling diaphragm piston is used to expand or retract the movable plate. Total combined area of the plates is 10 square inches. Outside diameter is 2.8 inches retracted and 3.7 inches expanded. Color-coded, high pressure nylon tubing connects the head to the console box. A threaded stud at the upper end of the pull yoke is used to attach the head to the pullrod.

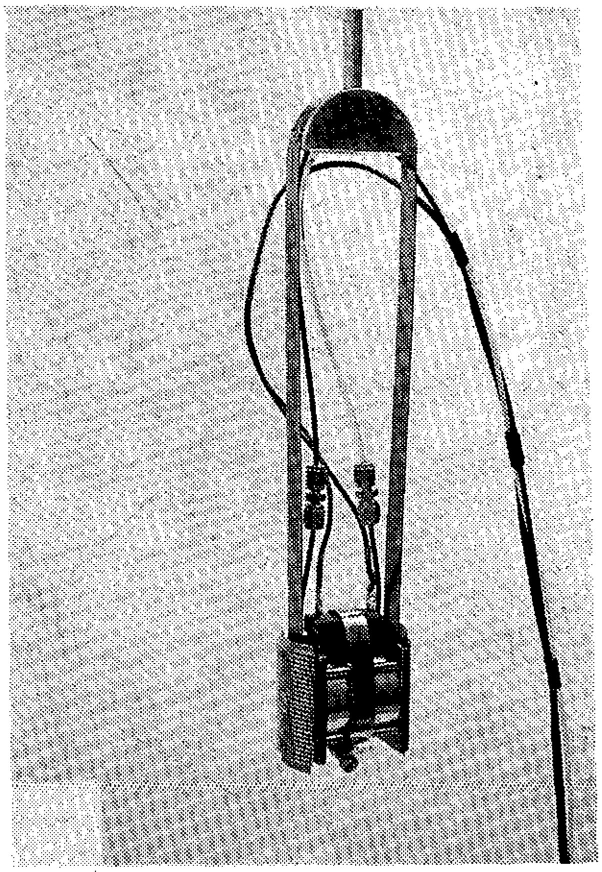


Figure 23 BOREHOLE SHEAR DEVICE (IOWA)

A handcrank-operated worm and pinion gear operates a vertical jackscrew. The pullrod extends through and engages a screw through a notched collet block. A bail on the block rests between the uprights that carry the handcrank and prevents the pullrod and jackscrew from turning. Therefore, operation of the crank places a tensile load on the pullrod and pulls the shearhead upward.

The pulling force is transmitted through a ball thrust bearing to a crossbar resting on two hydraulic loadcells. Pressure in the cells is read on a precision gage and converted to shear stress in pounds per square inch (psi) by use of a calibration graph chart.

The pressure console houses a small CO_2 cylinder with a shutoff valve. High pressure gas is fed to the regulator which controls pressure to the desired value of normal stress. Regulator output flows through a two-position "Retract-Expand" valve. A precision gage indicates pressure in 2 psi increments. Quick connect fittings are used to plumb the shear head to the control box. Connections are identified and color-coded to aid in correct plumbing to the shearhead.

Pullrods are furnished with the kit by the manufacturer in 2.5 foot sections, sufficient for testing a 45 foot depth. Rod sections are flush-jointed by threaded ends. Grooves are machined at one-half foot intervals to aid in measuring depth of test.

A special hand auger is also supplied with the kit. Sections of pipe and a tee handle are used to assemble the auger. A hole can be hand-drilled to shallow depths in cohesive soils. The cutter head may also be used as a temporary casing. The shearhead can be placed within the shell, and expanded in place under low pressure. Sections of pullrod and pipe are then assembled to the desired length. After lowering to near test depth, the shear plates are retracted and head advanced clear of the shell.

The vise plate is used at the top of the hole to support the shearhead while lengths of rod or pipe are added, and to prevent accidental dropping of the head. The vise plate and worktable are stored separately from the borehole device.

Modification and Adaptation

The setup supplied by the manufacturer required that the operator be in a squat position to conduct the test. The total time required for conducting a series of six tests at a given elevation is about forty minutes, which was extremely tiring to the operator. Hence, a worktable was designed which can be used on slopes as well as level ground to permit testing in the standing position (see Figure 24). It was found during field evaluation that the shearhead is likely to be overextended during operation. To avoid this, a warning circuit was installed consisting of a modified micro switch (at the shearhead) wired to a buzzer alarm. When the shearheads are extended approximately 1/2 inch from the retracted position the buzzer sounds, warning the person conducting the test.

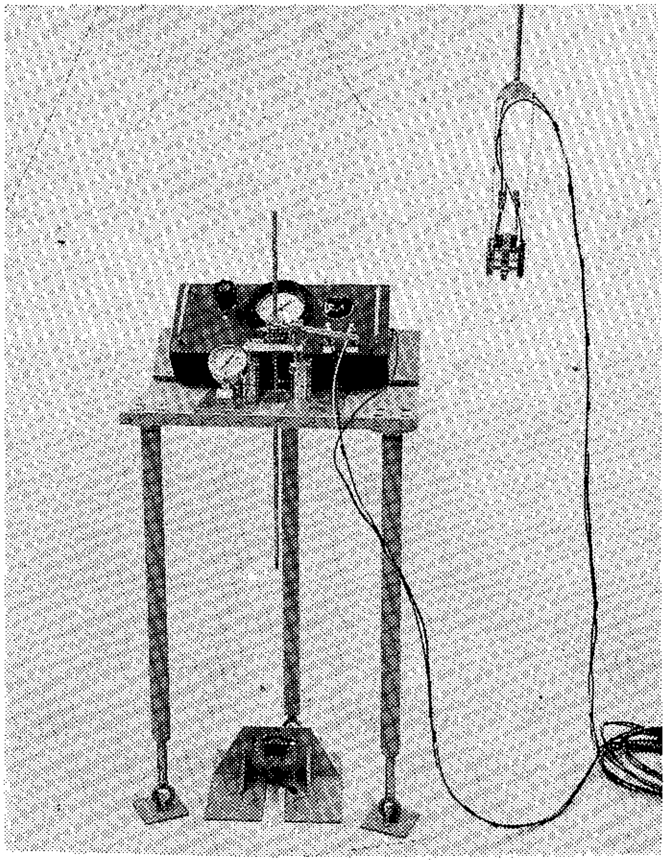


Figure 24 WORKTABLE FOR BOREHOLE SHEAR DEVICE

Operational Problems

An important factor in arriving at the correct values of cohesion and friction angle is proper seating. Lacking this, the first point becomes unreliable. For example, if the soil is hard and the normal pressure is too low for quick seating, the testing time will be prolonged and an initial false angle of friction will be obtained. As the seating occurs with successively higher normal pressures, the soil area contacted and brought to shear is approximately proportional to the normal stress. On the other hand, if the soil is soft and the increment of the normal pressure is too large, only one or two points can be obtained before the test apparatus is fully expanded, as indicated by a reduction of maximum shear stress at higher values of normal stress. While the first two points establish a straight line, a minimum of three, and as many as five or more, are preferable for statistical treatment of the data.

If gravel is encountered during the shear, the shearing stress will rise rapidly and proportionately as the crank is turned. Also, if cranking is stopped, no relaxation will occur. The indicated shearing stress will be much too high and may reach the gage limit of the apparatus, in which case the test is aborted. In other instances, the gravel particle may be turned aside and subsequent points will be on the true failure envelope.

In some instances it was noted that the probe was extended to its limits before the three to five test points could be obtained due to excessive disturbance on the sides of the borehole. This problem can sometimes be overcome with the use of drilling mud. The shear plates of the probe will press through the mud and, from the second test on, will give good values. As previously explained, in order not to overextend the probe, a buzzer circuit was installed to provide warning that the test limit was being reached.

Limitations and Advantages

In sands and silts the probe gives values of cohesion and friction corresponding to a consolidated-drained triaxial shear test. In clays, the results correspond to a state between partly drained to undrained conditions due to the changing pore pressure in relation to the normal stress.

In soils where caving or squeezing is expected to occur, the probe cannot be used. In some cases, the use of drilling mud will provide a solution to this problem. In hard soils, the shear plates do not seat during application of normal pressure. The heads of the shear plates tend to scrape over the surface of the soil so that the value of the apparent friction angle is observed to be very high. In some cases, the result may be a low (and often negative) cohesion intercept. Therefore, these soils cannot be tested with the conventional borehole shear device. The maximum testable cohesion is of the order of 8 psi or 0.6 tons per square foot.

When drilling mud is used, the shear plates pick up soil from the sides of the borehole. In this case, the test data may reflect unrealistically low shear strengths with no friction angle.

The borehole shear test permits the rapid calculation and plotting of the results with the equipment remaining in position. In the field, the operator can often determine whether he has attained the full consolidated drained or full consolidated undrained condition by study of the progressive sequence of his data points from one location.

This is possible because this technique involves the performance of stage testing which has the advantages of saving time and not requiring the often unobtainable duplicate specimen otherwise needed for laboratory testing.

Even though this test is conducted in the smear zone of the borehole, soil disturbance does not seem to affect the angle of friction values. This is true more in the case of consolidated drained tests than unconsolidated undrained tests. A more significant effect of soil disturbance is a reduction in cohesion values in consolidated drained tests which is due to destruction of natural cementation in the smear zone. In contrast to this, in a consolidated undrained test the effect is to increase the value of cohesion.

The Iowa Borehole Shear Device test has certain advantages. The shear tests can be conducted at the site. They can be conducted rapidly and with very reliable results if prior knowledge of the soil profile will permit testing in preselected weak areas. At the same elevation, a 90° rotation of the shear heads permits a second determination of strength values. Also, it should be noted that "undisturbed" sampling is not required to obtain the shear parameters. The Mohr envelope can be obtained using this device in considerably less time and at considerably less expense, than by the normal laboratory procedures. The test is simple in its nature and is expected to gain wider acceptance. Handy(4) summarizes the main advantages of the borehole shear test as follows:

> "1. Separate identification of c and ϕ in perhaps one-tenth the time required for laboratory triaxial or direct shear testing, enabling use where the laboratory test costs would be prohibitive.

2. Tests possible in hand-bored as well as machine-bored holes, where machine access is difficult or impossible.

3. Test data plotted on-the-site during the test, enabling immediate repetition if results are not reasonable.

4. Each test is conducted in a thin (3 inch) layer previously identified by boring. The test thus should deal no surprises, but should serve to quantify expectations.

5. The test is particularly appropriate where it is difficult or impossible to "blind sample" three triaxial samples from the same layer.

6. Ready statistical evaluations of variable soils."

Application to Geotechnical Problems

If settlement is not a critical problem, this test can be used to determine the ultimate foundation bearing capacity. It is extensively used by the Kansas State Highway Commission for design of foundations for light poles to withstand 100 mile per hour winds. The probe is also routinely used for laboratory determination of cohesion and friction of soil contained in standard compaction, on California bearing ratio (CBR) mold samples drilled and tested while in the mold. It has also been used for the design of foundations for transmission towers, and in the evaluation of active landslides where the factor of safety is known to be 1.0. The weakest layer can be tested even if it is only a few inches thick.

CHAPTER 5. VANE SHEAR DEVICE

Operational Concept

This device, originally developed in Sweden, evolved into its present modern form in the late 1950s. Arman, Poplin, and Ahmad(5) traced the history of development of the vane shear device in the following paragraphs:

> "The vane shear device was developed independently in Sweden and Germany during 1928 and 1929. However, it was not put to serious use until the Swedish Geotechnical Institute systematically began to study its reliability and effectiveness with a series of theoretical and practical tests in 1947. The results were presented in a classical paper by Cadling and Odenstad(6). Since then, the vane shear device has been frequently used and studied.
>
> "Skempton(7) described a vane (developed in England) for testing soft clays. Later, Bannet and Mecham(8) and Gibbs, et al(9) developed similar devices. Andresen and Bjerrum(10) described a vane borer that could also be used for penetration tests.
>
> "Later on, a vane borer was developed at Chalmers University of Technology in Goteborg, Sweden, that did not utilize a casing. It had, as a unique feature, a loose angle of "slip" coupling, which allowed the separate determination of frictional and shear resistances. The unit was equipped with a recorder using pressure-sensitive paper.
>
> "The application of torque through a worm gear assembly and the use of a recorder to isolate friction and shear failure modes appears to be the best available combination on a vane shear device Osterberg(11).....

"The relationship between undrained shear strength, s_u, and torque was developed by Cadling and Odenstad(6) in the form of

$$s_u = \frac{6}{7} \frac{T}{\pi D^3}$$

where T = torque, D = vane diameter, the ratio of vane height to vane diameter is 2.0.

Skempton(12) later observing that the failure surface had a larger diameter than that of the vane, modified the above relationship to include an effective diameter, XD, with X taken to be 1.05. Flaate(13) suggested that the undrained shear strength could be determined by assuming the shear strength at the ends of the sheared block to be mobilized proportionately to the strain. He modified the equation to

$$s_u = \frac{8}{9} \frac{T}{\pi D^3}$$

"The effects of the vane shape and size were investigated by Flaate(13), Osterberg(11), Eden and Hamilton(14), Andresen and Sallie(15), and others. The area ratio of a vane is defined as the ratio of the cross-sectional area of the vane to cross-sectional area of the sheared cylinder, which is directly proportional to the vane size. Thus it was necessary to find an optimum area ratio. Earlier investigators agreed that the smaller area ratio will cause less disturbance. Ratios recommended by various authors ranged between 10% and 25%.

"The work done in Sweden indicated that a height to diameter ratio of 2 produced consistent results. Cadling and Odenstad(6) showed that, when an H/D ratio of 2 is maintained, the vane diameter has no effect on the results. This ratio has been accepted by most manufacturers of vane shear devices as universal.

"Osterberg(11) and Bazet, et al(16) reached similar conclusions. However, in muskeg tests, Northwood and Sangrey(17) found the smaller vanes to give greater scatter in the results than the larger vanes.

"Vane shear devices have also been used to determine the sensitivity of soils. The remolded vane shear strength has been determined in-situ after giving the vane several full turns [Skempton(18); Fenske(19); Eden and Crawford(20); Bazet et al(16); and Serota(21)].

"Fenske defined this strength as minimum shear strength.

"Eden and Hamilton(14) found disagreement between the sensitivity of Leda Clays as measured by complete remolding and that measured by vane shear tests.

"The differences in shear strength obtained with the field vane shear device and unconfined compression and triaxial tests have been explored by many investigators. Several relationships, statistical and otherwise, have been established [Carlson(22); Skempton(12); Bannet and Mechan(8); Aldrich(23); Osterberg(11); Fenske(19); Andresen and Bjerrum(10); and Anderson et al(24)]. The relationships vary according to soil type as well as environment."

NOTE: Reference numbers have been changed.

The vane shear test consists of placing a four-bladed vane in the undisturbed soil and rotating it from the surface to determine torsional force required to cause a cylindrical surface to be sheared by the vane. This force then is converted to a unit shearing resistance of the cylindrical surface. It is of basic importance that the friction of the vane and the instrument be accounted for; otherwise, it would be improperly recorded as soil strength. Friction measurements under no-head conditions are satisfactory only if the torque is applied by a balanced moment that does not result in a side thrust. As torsional forces become greater during a test, a side thrust in the instrument will result in an increase in friction that is not accounted for by initial, no-load conditions.

The test has been adopted throughout the world because of its usefulness and established reliability. It is designed primarily for determining the undrained shear strength of the in situ fine-grained soils of soft to medium consistency and free of sand layers and pebbles.

Description of Device

The standard dimensions of vanes are given by ASTM Designation D-2573-72 ("Standard Method for Field Vane Shear Test in Cohesive Soils"(25)). The vanes used in this research project are shown in Figure 25. Each basically consists of four thin blades welded cross-like to a small cylindrical shaft. The size of vanes most commonly used range from two to three inches in diameter, and four to six inches in height. A height (H) to diameter (D) ratio of two has been generally recommended. The size selected for a particular application depends on the estimated strength of the material and the limitations of the torque device at hand.

Extensions to the torque shaft are attached as required. The vane can function satisfactorily to depths of about 50 feet. As the shaft must be protected by an outer casing against soil friction and guided by bearing, friction caused by the bearing must be allowed for in the calibration process.

Torque can be applied to the vane in several forms, ranging from a simple spring balance device to an elaborate worm gear mechanism (Figure 26). By means of a small crank with a worm gear, the torque can be applied at the proper speed (.1 degree per second) by turning it at one revolution per second. A direct reading calibrated dial gives the torque directly. Three types of vanes (Figure 25) were used in this research effort, and their dimensions and height to diameter ratios and area ratio are tabulated in Table 3.

TABLE 3 - VANE SPECIFICATIONS

	H	D	H/D	Area Ratio
3"-Tapered	8.25"	3"	2.75	0.10
2"-Tapered	5.45"	2"	2.73	0.21
2"-Square	4.0"	2"	2.00	0.21

The maximum depths to which the vane tests were conducted and the different types of vanes used in different sites are presented in Table 4.

TABLE 4 - DETAILS OF VANE TESTS

Site	Boring	Vane	Maximum Depths Reached (ft)	No. of Tests
1	V-1	3"-Tapered	9.5	1
1	V-1	2"-Tapered	21	5
1	V-1	2"-Square	14.5	1
2	V-2	2"-Square	41	6
3	V-1	2"-Square	19	2
3	V-1	2"-Square	24.5	2
3	V-2	2" Square	12.5	1

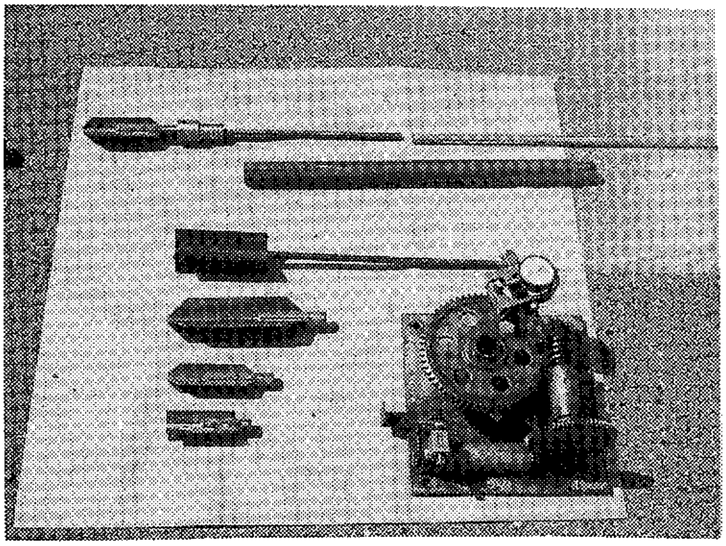


Figure 25 VANES USED

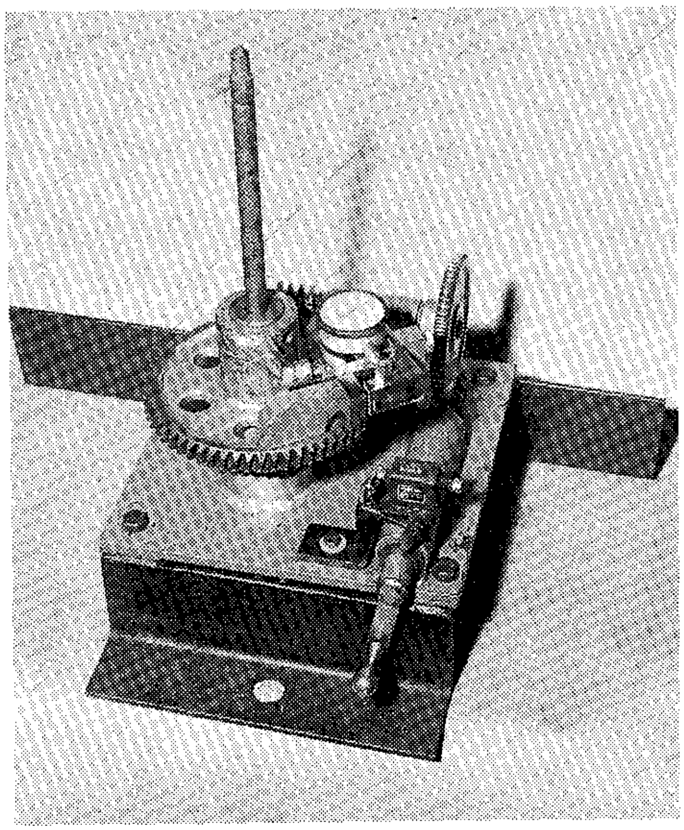


Figure 26 WORM GEAR MECHANISM TO APPLY TORQUE

From these two tables, the following can be noted. Whereas, a height to diameter ratio of two is recommended in the ASTM specifications, in this research effort ratios ranging from 2.0 to 2.75 were used; whereas, an area ratio of 0.1 was recommended in the ASTM methods, ratios varying from 0.1 to .21 were used.

Operational Problems

The major problems encountered during testing include: frictional resistance affecting the vane rod; poor alignment of the rod assembly, which results in a side thrust; and failure to retract the vane prior to advancing the rod to the next test elevation. Other problems are summarized as follows:

1. Loss of vanes; and bending or shearing of vane adapter rods caused by the vane not being retracted after a test and prior to advancing the "A-rod" housing to the next test depth.

2. The mounting table or platform on the B-61 Mobile drill rig which supports the vane shear torque head is a movable sliding support used for other types of drilling and testing operations. When used as the torque head vane support, it must be secured in place to prevent slippage during vane tests.

3. The ASTM test procedure for vane testing requires "remolded strength tests" immediately after running the in situ test with the vane. The first step is to rotate the vane "rapidly" for 10 revolutions and begin the test within one

minute thereafter. Since it requires 3600 revolutions of the actuating crank to rotate the vane rod 360 degrees, it was impractical to conduct the remolded test. A drill fitted with a shaft adapted to the crank opening on the vane shear torque head could possibly be used to accomplish the 10 "rapid" revolutions of the vane. If this method is adopted, the revolution counter should be disconnected prior to engaging the drill. For this project, a removable crank was fitted to the vane rod, and the vane was rotated 10 times by hand.

Limitations and Advantages

This probe cannot be adapted to varying soil conditions. It is limited to use in soft clayey material with shear strengths less than one ton per square foot.

It is traditional to assume the failure surface as cylindrical. When the vane is rotated within the soil, the failure surface induced has been proven by radiographs to be larger than the cylindrical volume assumed in theoretical calculations. In fact, the diameter of the mass of failure is found to be larger than the diameter of the vane.

A certain amount of soil disturbance takes place when the vane is rotated within the soil mass. Cadling and Odenstad(6) conducted research on disturbance around the vane blades. They and other researchers have found that progressive action takes place during the shearing induced by the vane. The degree of disturbance induced by the blades and the progressive action are two factors that have been found to affect the shear strength of sensitive clays.

There are many other factors that affect the test results. These include: effects of rate of penetration, progressive action, the thickness of the vane blade edge, strength and anisotropy, and pore pressure effects. Further research is needed to quantify the effect of these factors on the shear strength values obtained. Other factors that can affect test results include disturbance, rate of rotation, and delay in start of the test.

Among its advantages, the probe is portable and it is very simple and economical to operate. Many researchers have found that this probe gives comparable values to those obtained from unconfined as well as triaxial shear strength tests. Again, the experience of quite a few of the researchers who have used various kinds of vanes shows that the test results are fairly repeatable.

Studies by Arman, Poplin and Ahmed(5) have indicated that the effect of the shape and size of the vane on the measured shear strength is insignificant.

Application to Geotechnical Problems

Until quite recently the vane shear test was assumed to give a direct and accurate measure of in situ undrained shear strength. However, recent indications are that the undrained shear strengths obtained using this probe are unrealistically high. Using this theme, Bjerrum(26) attributed the problem primarily to rate of strain effects. Bjerrum suggested a correction for vane shear strength data based upon Plasticity Index. Ladd and Foote have subsequently added a few more data points to Bjerrum's correction curves(26).

It has been shown conclusively that the shear vane undrained shear strength values obtained with weak sensitive clays are less than those from compression tests on block samples. This is attributed to progressive action effects due to the varying thicknesses of vane blades.

Although many researchers have found that sensitivity has very little practical meaning, good correlation was obtained in this research effort between values of sensitivity obtained with the vane shear probe and those using test data from unconfined compression tests as well as laboratory triaxial tests. Other researchers have found that the vane shear clay sensitivity usually exceeds the sensitivity values obtained using laboratory tests. This may be due to the fact that vane tests induce very large strains at failure which do not represent field conditions.

CHAPTER 6. DUTCH CONE PENETROMETER

Operational Concept

Various kinds of penetrometers have been used around the world. Static cone penetrometers have been used extensively in Europe since about 1930. The standard penetrometer has been more widely used in the United States, although the static cone is rapidly gaining in popularity in this country.

In static cone penetration, a stage is reached during the test when the friction along the rod becomes a significant part of the driving resistance so that the measured resistance is no longer representative of the resistance beneath the cone. In order to eliminate this element of resistance, a patented system has been developed in Holland in which the cone has been placed in a tube. This system, known as the "Begemann's Sleeve", was used on the project. The cone and the sleeve are alternately pushed into the soil. The readings taken permit a determination of the resistance due to friction along the sides of the sleeve as well as cone penetration resistance.

The main tube with the pressure rod inside and the cone assembly at the lower end is pushed at a uniform speed vertically beneath the soil surface, usually to a depth of 20 centimeters. The pressure rod extending above the tube is then pressed slowly down another 5 centimeters. The force required to overcome the resistance encountered by the sounding cone, commonly referred to as the cone resistance or the cone bearing capacity, is read on the pressure gage. The pushing of the tube followed by the cone is repeated regularly every 20 centimeters of depth. The results are then plotted in the form of a depth profile; the depth being represented on the y-axis, and the cone reading and other data on the x-axis. A cone with a base area of 10 square centimeters and slope of 60 degrees has become the standard of the industry.

Description of Device

The two main sources of information on cone penetrometers are Sanglerat(27) and a report of the Federal Highway Administration by Schmertmann(28). The description in this section will be basically limited to the device that was used in this project. For information on other penetrometers and on methods of application to practical geotechnical problems the reader is referred to the sources noted above.

The static cone penetrometer often referred to as the Dutch Cone Penetrometer was used in this evaluation. It was developed in Holland through the combined efforts of the Delft Soil Mechanics Laboratory and the Goudsche Machinefabriek Company.

The standard Dutch Cone equipment consists of two cones. The first is called the jacket cone while the second is referred to by the name of its inventor, the Begemann friction jacket cone (Figure 27). The specifications for the cones, friction reducing tube, sounding tubes, inner rods, and the load cells with gages are reproduced in Figure 28.

The standard Dutch jacket cone is a steel 60° right circular cone with a diameter of 3.65 cm and a projected end area of 10 square centimeters. The standard Dutch Begemann friction cone is also a steel 60° right circular cone with a diameter of 3.65 cm. A smooth steel friction sleeve is mounted directly above the cone in a manner which will allow the cone and the sleeve to extend and collapse in a vertical direction by means of a double rod system, as shown in Figure 29.

The penetrometer rods consist of an outer tube of 36 mm outer diameter and 16 mm inner diameter, with a solid inner rod of 15 mm diameter. Both outer tube and inner rod are one meter in length. The outer tube is fitted with tapered threads for connecting individual sections to make up the necessary testing lengths. The top section of the penetrometer is threaded to provide a coupling with the outer tube. The inner rods have squared ends and are stacked end to end inside the push tube.

Operational Problems

It was found during the field operations that the sounding rods above the top of the hole had a tendency to bend. In the ASTM procedure, the tubular rod guide is recommended. In this research project, a five-foot length of hollow stem auger was used; being driven from the ground surface to a depth of about 5 feet and left in place. This prevented bending of the rod for the most part; although, when the length of the sounding rod was greater than 30 feet, it was found to bow slightly.

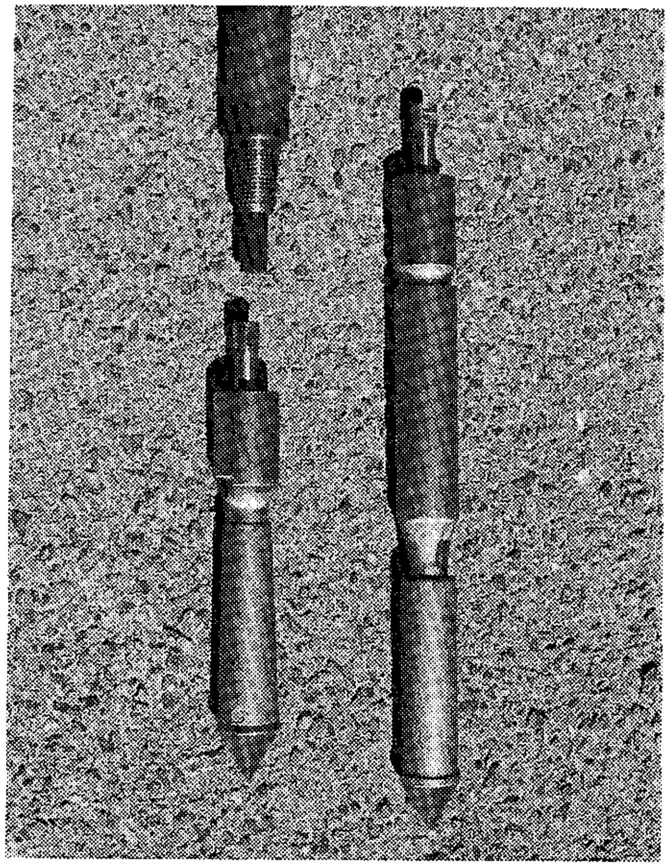


Figure 27 DUTCH CONE PENETROMETERS

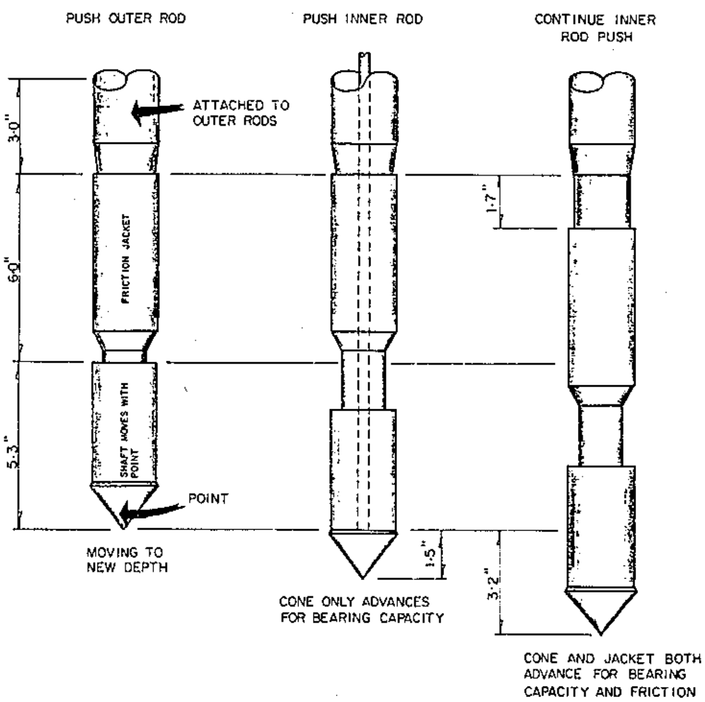


Figure 29 SCHEMATIC DRAWING OF BEGEMANN FRICTION CONE AND EXTENSIONS DURING TESTING

Sounding Gear

A) Jacket cone; max admissible load 7.000 kgf
weight of jacket and rod: 0.4 kg
cone base area: 10 sq.cm.
apex angle: 60°
stroke: 70 mm

B) Friction jacket cone max. admissible load: 7.000 kgf
weight of friction mantle + rod: 0.8 kg
cone base area: 10 sq.cm.
apex angle: 60°
stroke of cone: 35 mm
stroke of cone plus mantle: 35 mm
total stroke: 70 mm
surface of friction jacket: 150 sq.cm.

C) friction reduction section tube

D) sounding tubes:
seamless: Ø 36/16 mm x 1000 mm long
weight: 6.65 kg

E) inner rods:
Ø 15 mm x 1000 mm long
weight: 1.40 kg

Load Cell With Gages

area of the measuring plunger: 20 sq.cm.
measuring range of the low pressure gage 0 - 100 kgf/sq.cm.
measuring range of the high pressure gage 0 - 600 kgf/sq.cm.
the low pressure gauge is protected by an automatic shut off valve to overload.
indications on the low pressure gauge over 80 kgf/sq.cm. are therefore not exact.
weight of the pressure sleeve: 1.65 kg
weight of the pressure hammer + handle: 0-40 kg

From Ref 29

Figure 28 SPECIFICATIONS FOR DUTCH CONES

It was also found that keeping the sounding rods with the penetrometer absolutely vertical was extremely important. Otherwise, the sounding rods had the tendency to bend and start drifting. Drifting can be reduced by using rods which are initially straight and by making sure that the initial cone penetration into the soil does not involve unwanted initial lateral thrust.

The manufacturer supplied a tube called a lateral friction-reducing section tube. Its purpose was to increase the penetrometer depth capacity. This special rod, which has an enlarged diameter or special projections, is attached to the main rod just above the cone penetrometer. When it is pushed with the rest of the sounding rods into the ground, it creates a hole having a diameter larger than the diameter of the cone. This causes the reduction in friction and thus increases the penetrometer depth capability.

Limitations and Advantages

With routine or conventional soil exploration, undisturbed samples will be recovered. When soil exploration is done with the static cone penetrometer, soil samples are not recovered for visual inspection. Another limitation is the depth of penetration due to the thrust capability of the standard equipment. With the help of the friction-reducing section tube, a few more feet of penetration can be achieved.

The main advantages of the cone penetration test are its speed, simplicity, and economy.

The Dutch Cone provides test data that are more amenable to analytical interpretation than those obtained by the standard penetration tests. This statement should, perhaps, be qualified by the results of the recent study of Schmertmann on correlation data from the standard penetration test with the Dutch Cone tests.

Applications to Geotechnical Problems

The static penetrometers have already been widely used to investigate the properties of soil deposits in situ. They have been used extensively in Europe since about 1930. The geotechnical applications have to be historically divided into two categories: studies done before 1970, and those since. The studies prior to 1970 are summarized below by Durgunoglu and Mitchell, 1975(31).

> "Previous studies have shown that static cone penetration test results can be used:
>
> "(1) To derive information on soil types and soil strength; e.g., De Beer(32), Plantema(33), Kondner(34), Meyerhof(35,36) and Begemann(37,38).
>
> "(2) As a basis for determination of pile supporting capacity, as shown for example, by Van der Veen(39), Bogdanovic(40), Kerisel(41), Menzenbach(42) and De Beer(43).
>
> "(3) To estimate compressibility and in situ density of cohesionless soils; e.g., Meyerhof(35), Rodin(44), Meigh and Nixon(45), Schultze and Melzer(46), and Bachelier and Parez(47). There is currently considerable interest in the deduction of relative density values of cohesionless soils from cone resistance data for use in the assessment of liquefaction potential.
>
> "(4) For estimation of the settlements of footings on sands, according to the methods of Buisman(48), De Beer and Martens(49), Schmertmann(50), and others.

"(5) To characterize vehicle trafficability over unpaved soils, as proposed, for example by Murphy(51), Freitag et al(52), and Wiendieck(53).

"None of the analytical solutions for penetration resistance that have been developed (e.g., De Beer(32), Meyerhof(36), Biarez and Gresillon(54), and Vesic(55) can account simultaneously for relative depth, size, soil friction angle, soil compressibility, in situ lateral stresses, cone angle and cone roughness, all of which are known to influence the ultimate cone resistance. In addition, some of them do not consider soil cohesion and, therefore, they are applicable only for cohesionless soils."

CHAPTER 7. STANDARD PENETROMETER

Operational Concept

In the United States, the most commonly used penetrometer is the ordinary split sampling spoon. Penetration occurs as a result of the dropping of a 140-pound hammer onto the drill rod from a height of 30 inches. The number of blows necessary to produce a penetration of one foot is the penetration resistance of the soil. The procedure is based on measuring the resistance offered by the soil to the advancement of the penetrometer. As discussed in Chapter 6, if the penetrometer is pushed steadily into the soil the procedure is called the static penetration test. If driven into the soil, it is referred to as the dynamic penetration test. Although static tests are preferable in connection with soft cohesive deposits, dynamic tests may be useful in very hard deposits. Both static and dynamic tests are useful in cohesionless soil deposits.

Description of Device

Three main components are necessary to conduct the penetration tests in addition to the accessory equipment: the drilling equipment to conduct the test; the special split barrel sampler; and the drive weight assembly. Accessory equipment consists of test forms, sample jars, paraffin, and other miscellaneous supplies.

The drilling equipment used in this research project was a Mobil B61. It is well designed for conducting penetration type tests. The drill rod should have a thickness equal to, or greater than, the A rod. For depths greater than 50 feet, a different drill rod is recommended. The split barrel sampler and drive shoe should be hardened steel (Figure 30). The drive weight assembly consists of a 140-pound weight, a driving head, and a guide permitting a free fall of 30 inches.

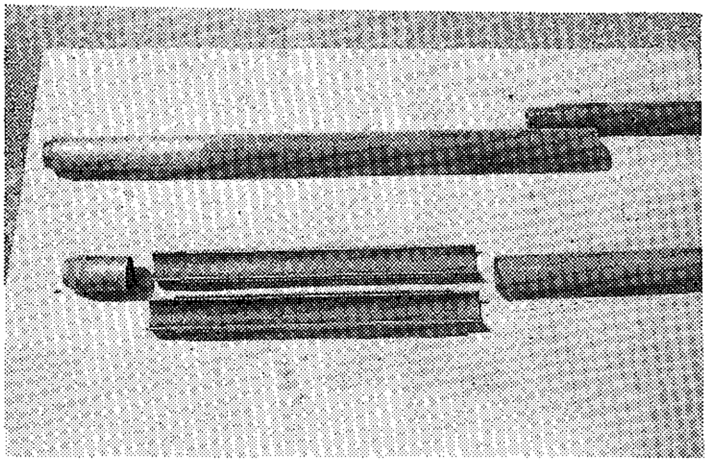


Figure 30 CALIFORNIA SPLIT BARREL SAMPLER

Modification and Adaptation

The term "cat head" is driller's jargon for the revolving drum or spindle located in a convenient place, usually at the right rear of truck-mounted machines. It is basically used for lifting heavy items and has been frequently used in standard penetration operations for many years. It is manually operated by throwing several loose loops of large diameter hemp rope over the revolving drum. When lifting is desired, the driller pulls the trailing end of the rope, taking up the slackened loop which binds against the revolving steel surface causing additional takeup of the rope around the drum. The other end of the rope passes from the "cat head" over the ground block then vertically downward over the boring location. In the case of standard penetrometer tests, the 140-pound hammer is tied to the end. When the driller tightens the rope by pulling, the hammer is raised and the driller then releases sufficient slack in the rope to allow the hammer to fall 30 inches. The method is fast and reasonably efficient.

Operational Problems

Reproducibility of data has been found to be very difficult to achieve, due largely to lack of control. Failure to lift and release the 140-pound hammer exactly 30 inches has been a source of error from the beginning of this test. In recent years, researchers have undertaken to determine the results of test variations. The building industry has made attempts to develop automatic tripping devices that would insure a more precise length of free fall. Some progress has been made in this direction but the margin of deviation remains relatively large.

Limitations and Advantages

It has been found that the standard penetrometer test has poor reproducibility and broad variability of test data.

Further, the vertical effective overburden stress has significant effect on the test blow count. Hence, many authors have recommended using a modified value for the blow count. The data that have been presented in the literature for almost 50 years do not reflect such corrections. Designers should use "past data" with caution and judgment. It has also been proven that horizontal stresses at various depths within the hole have an effect on the blow count value.

There is a need for free-drop control. The ASTM test procedure does not specify a way by which the operator might achieve this. Several studies have indicated that the typical cat head and slackened rope procedure cause a major increase in the value of the blow count. It would seem that the hammer drop method - free of operator variation - would be most reproducible. This technique has been adopted in various countries including England, Israel, and Japan. This test, as many of the in situ tests, does not take into account the seepage and drainage conditions in the hole.

Some engineers do not even refer to standard penetration as a "test". For up to 50 years, the test data derived from standard penetrometer have been used on an empirical basis. During the last few years, a few researchers, notably Schmertmann(56,57) and Kovacs(58,59) have conducted extensive theoretical research on this subject.

The loose control of the standard penetrometer test procedure and equipment has discouraged some engineers from using it extensively. The test is considered by some not worth upgrading. In any case, the test data can be considered as an indication of the consistency or the relative density of the soil. It has been used for many years and will probably continue to have widespread application. Even though this device is one of the earliest and most widely used in situ probes for soil exploration, it will likely be replaced by the cone penetration test.

It should be noted, however, that standard penetration equipment is relatively inexpensive, simple to use and operate, and the test results are easily applied. For these reasons, it has been accepted in many countries throughout the world. In the interest of standardization of this test method and equipment, the ASTM committee has been working for improvement and refinement in the method described.

Sanglerat(60) summarizes the limitations of the standard penetrometer tests. He believes the word "test" is a misnomer. Many factors affect its reproducibility. The condition of the sampler (deformation, rust) will influence the test results. The same holds true for damage to the driving shoe. Location of the groundwater level also affects test values.

The elapsed time between the drilling of the hole and the taking of the blow counts is not standardized. The driving energy of the falling hammer is bound to be absorbed due to the elasticity and the flexibility of the rods.

Fletcher(61) has pointed up certain limitations of the standard penetrometer test. For example, variations in the distance drop of the hammer: usually not exactly 30 inches. And the size of the rods affects blow count. Friction present could prevent a free fall of the hammer. Human error in counting the number of blows or in measuring the depth of penetration is another source of error. An excessively high value of the blow count will be obtained if the sample is compressed during the driving process. On the other hand, test data will be of no value if the sand or the soil being tested is disturbed, as may be the case at the bottom of a boring which causes water to flow rapidly into the hole.

Applications to Geotechnical Problems

As this in situ test has been in use for nearly 50 years, it has been used in various stages of geotechnical design. In the following paragraphs, a summary of these uses is presented.

The test blow counts have been used to estimate the effective friction angle of a sand deposit. Using data collected by the Russians, DeMello(62) presented in 1971 a method for estimating the effective friction angle from blow counts. Knowing the blow counts and the overburden pressure in kilograms per square centimeter, the effective friction angle can be estimated.

Terzaghi and Peck(63) proposed a method of estimating the bearing capacity of soils using the blow count as well as the friction angle in degrees. They also developed charts to estimate bearing capacity factors and from them an estimation of the bearing capacity of soils for shallow foundations. The charts provide for surcharge and settlement conditions.

The blow count values have been correlated with unconfined compressive strength values of clay soils. The ratio of unconfined compressive strength to the blow count has been found to vary with the soil type. This ratio has a value of 4 for clay soils, 5 for silty clays, and 7.5 for silty, sandy soil. Of course, these ratios will vary regionally, and from country to country.

Blow count values have been used to estimate the relative density of sandy soils. The values of relative densities of sandy soil deposits at various depths in a particular site have been very helpful in the estimation of liquefaction potential of sandy soils.

A great deal of research has been conducted on both sides of the Atlantic comparing the standard penetrometer with the static cone penetrometer test. Approximate relationships between the relative density of fine sand and the angle of internal friction developed from the standard penetrometer test have also been developed. According to Meyerhof, the static cone resistance in tons per square foot for fine or silty sand is equivalent to four times the blow count. This number might vary from region to region.

Engineers have also often used blow count values as an approximate but expedient method to estimate bearing and friction capacity of soils. The reader is referred to Norlund(64), Fletcher(61), and Schmertmann(65), who have suggested various methods of using blow counts to calculate the bearing capacity of soils as well as the friction capacity.

In 1967, Bazaraa(66) developed and presented a thesis in which he tried to correlate between the effect of overburden pressure on the blow counts as well as the effect of submergence and standard plate load test results. Combining the test data from various sites, he developed a method for estimating settlements of shallow foundations on sand.

CHAPTER 8. CAMBRIDGE SELF BORING PRESSUREMETER

Operational Concept

Self boring is a technique by which a probe is inserted into the ground with minimal soil disturbance. The primary difference between the self boring and other boring devices is that it has the capability for self insertion. This chapter concerns the Cambridge Self Boring Pressuremeter. It will be referred to as the Cambridge Pressuremeter, or Cambridge Probe, to distinguish it from another probe marketed by the same company, called Cambridge Total Load Cell.

The Cambridge probe essentially consists of a miniature cylindrical tunneling machine that is jacked steadily into the ground. The soil entering the shoe is cut into small pieces by the central rotating cutter and is then carried to the surface by a flushing fluid, which is normally water. The fluid is pumped down the inside of the cutter rods and up the annular space between the inner and outer rods. A cylindrical membrane is fitted over the outside of the instrument which can be expanded against the undisturbed soil as in a conventional Menard Pressuremeter test. This neoprene membrane is protected by an outer flexible stainless steel sheath, in some cases. The radial expansion of the membrane is measured at its midpoint by three separate pivoted levers which are kept in contact with the membrane by spring cantilevers. The probe loads the soil radially by inflating the membrane using gas pressure.

It provides a plot of radial strain versus applied pressure from which the shear stress/shear strain diagram for the soil is determined by a simple graphical transformation. Several soil parameters can be obtained from the results of these tests.

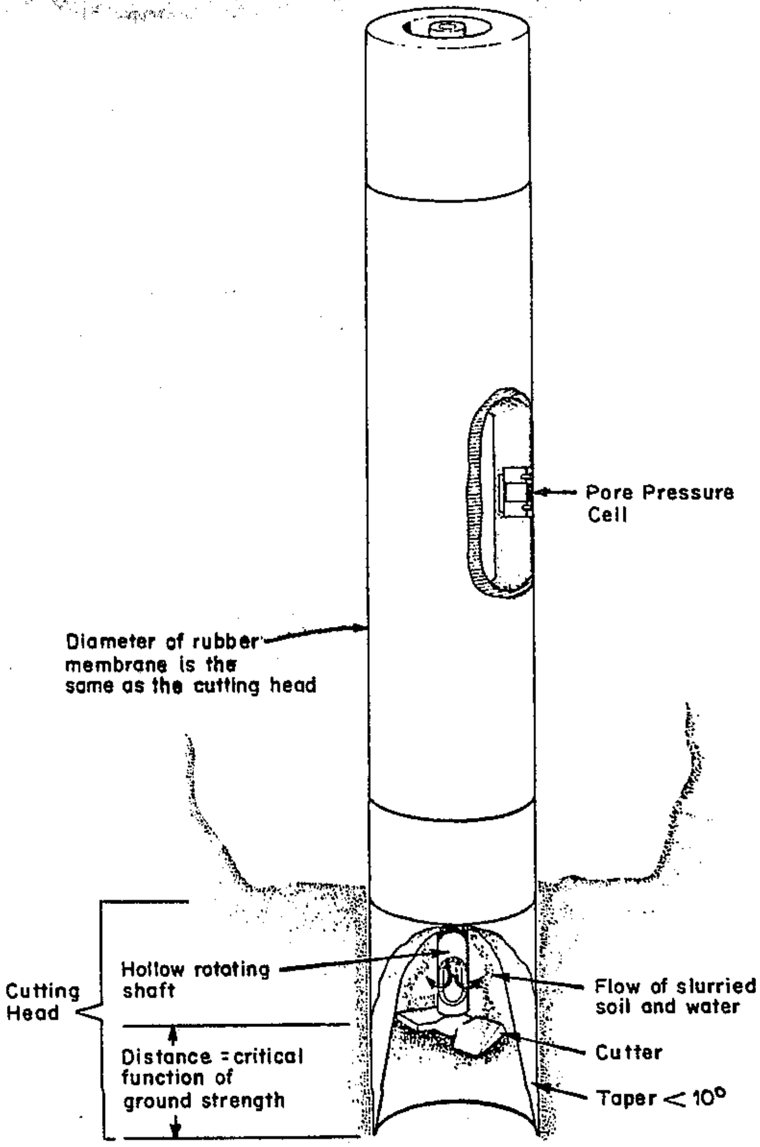
Description of Device

Self boring pressuremeters have three distinct and separate components. They are the driving module, measurement module, and the tunneling (self boring) module. In some devices, the driving module is separate from the measurement module and the tunneling module. Both probes designed by Cambridge In Situ Company belong to this category. These concepts are depicted in Figure 20 of Reference 2.

The expansion pressuremeter basically consists of the tunneling module and the measurement module, connected together and allowed to descend vertically in the hole by means of casings and rods. Schematic diagrams of the Cambridge pressuremeter before insertion and during an expansion test are presented in Figures 31 and 32.

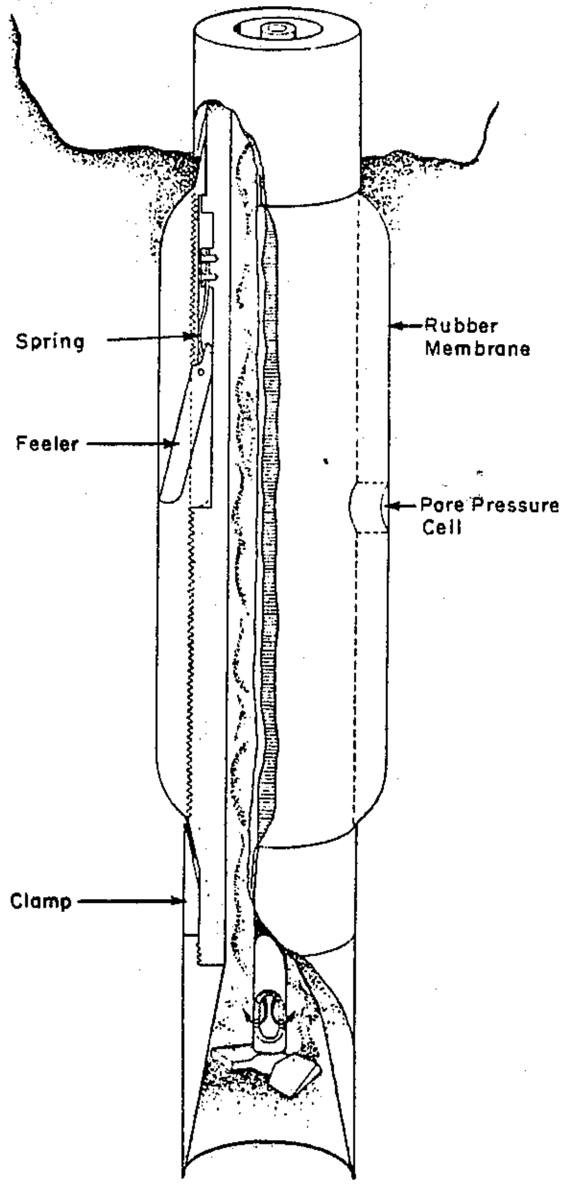
The probe cannot be used by itself. It requires a ground frame or a drill rig to function. All components needed to conduct a self boring pressuremeter test are listed below:

Probe
Drill rig or ground frame
Pump
Casing and cutter drive rod



From Ref 67

Figure 31 CAMBRIDGE PROBE BEFORE INSERTION



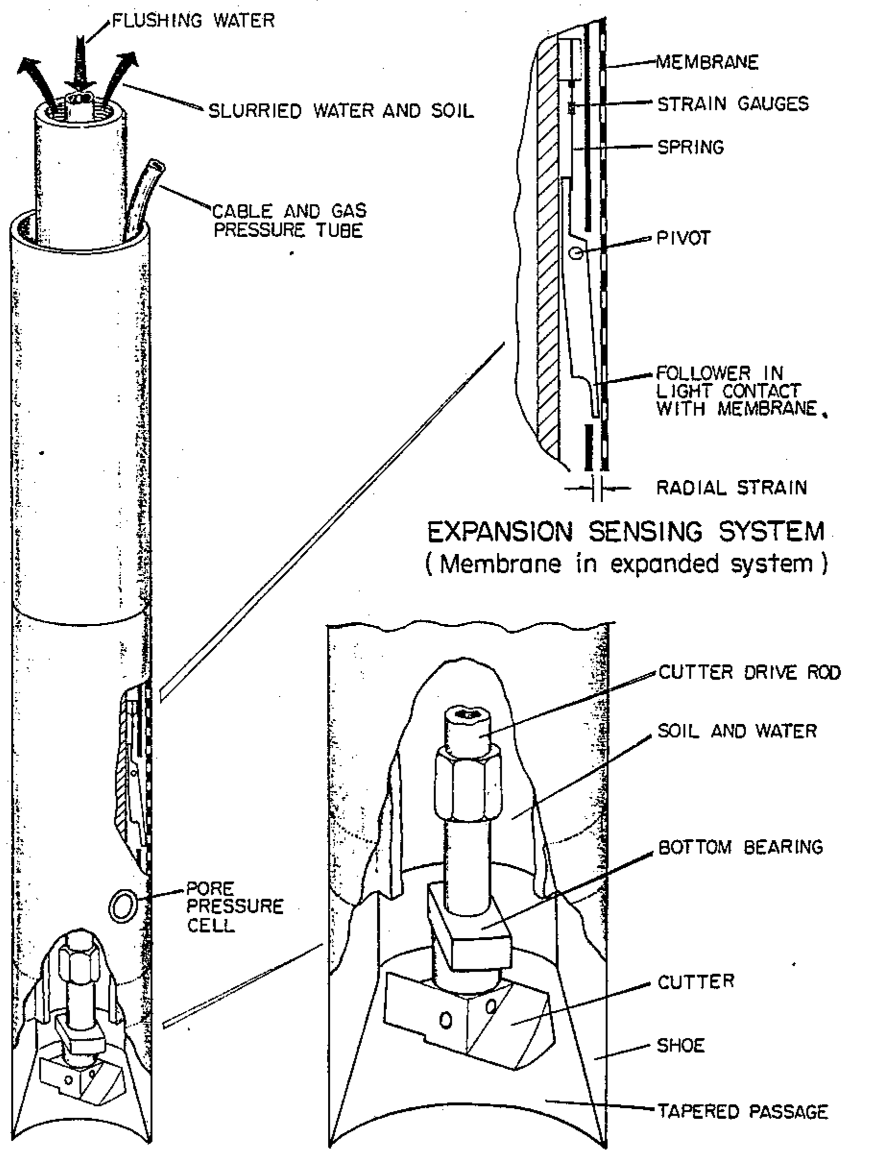
From Ref 67

Figure 32 CAMBRIDGE PROBE AFTER INSERTION

Strain control unit
Digital multimeter
X-Y-Y plotter
Gas cylinder
Vehicles to carry the probe and the accessories

As explained earlier, the probe itself usually consists of three parts. The two parts that go into the ground are called the self boring module and the measurement module. The self boring module is also called the low disturbance insertion system (Figure 33). It contains a hollow cutter drive rod for circulating water or drilling mud downward. The cutter drive rod is connected to the cutter, a sharp two-edged tool.

The cutter supplied by the manufacturer sheared repeatedly and was eventually lost in a borehole. Hence, Caltrans developed its own cutter design, as shown in Figure 34. Holes within the cutter allow water jets to impinge on the sides of the cutting tool. The measurement module consists of a rubber membrane slid over a portion of a cylindrical metallic cylinder. At its center it has three expandable radial arms in light contact with the membrane. This is the expansion-sensing system actuated by a supply of nitrogen oxide (Figure 33). When the membrane is expanded during the conduct of the test, the radial arms that are fixed at the center of this measurement module follow the membrane. These readings are measured by a strain-control unit and thus radial strain is measured. There are also two pore pressure cells fixed at 180° to each other for measurement of the pore pressure.



From Ref 67

Figure 33 LOW DISTURBANCE INSERTION SYSTEM AND EXPANSION SENSING SYSTEM

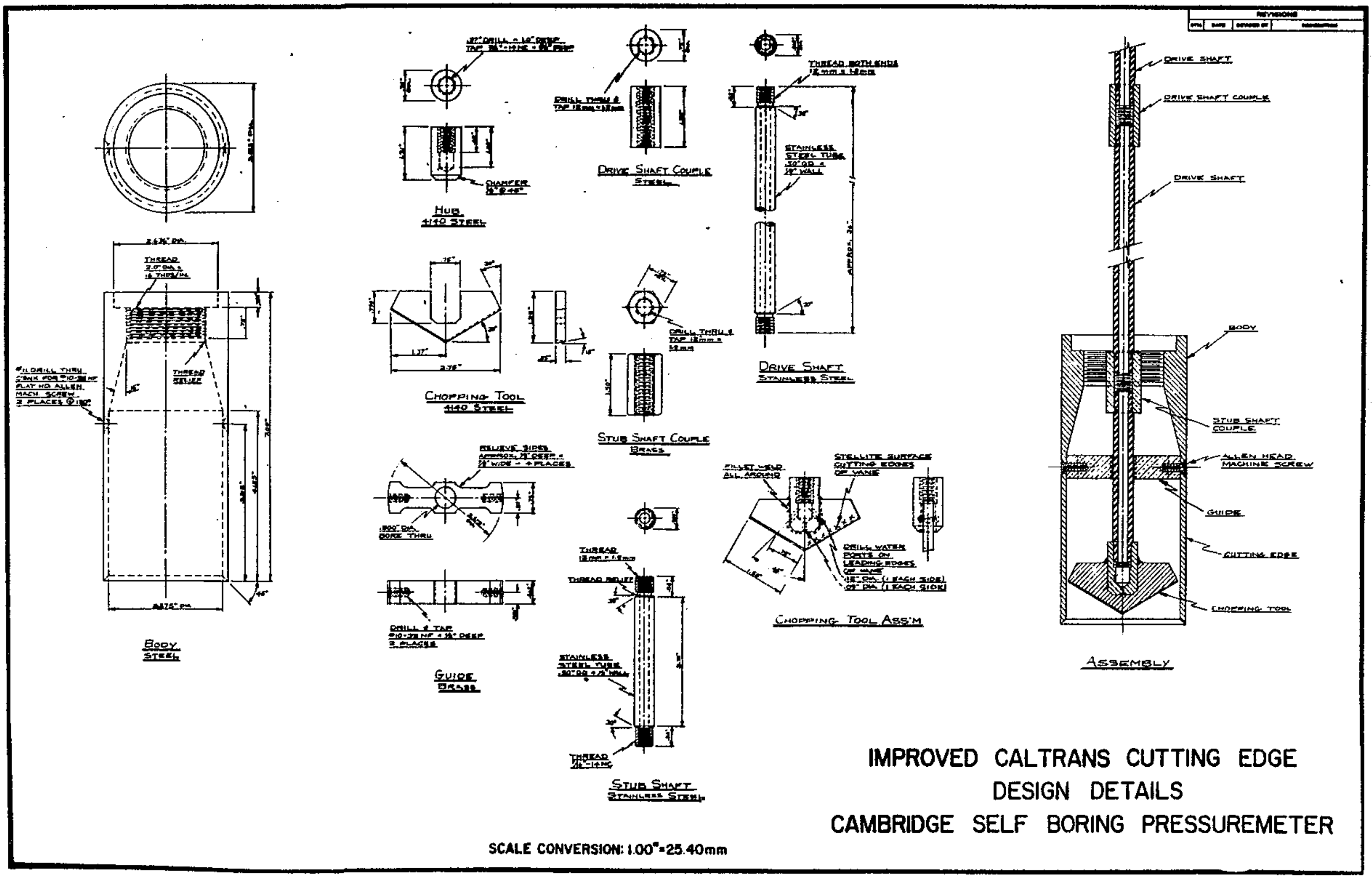


Figure 34 IMPROVED CALTRANS CUTTING EDGE - DESIGN DETAILS

The probe can only be used in conjunction with a drill rig or a ground frame. For this study a Concore N-69 drill rig was used. The hydraulic system of this drill rig required modification by installation of a hydraulic bypass valve. To rotate the cutter drive unit, a cutter drive mechanism was designed. The drawings that are necessary to manufacture a cutter drive unit are presented in Figures 35 and 36. The cutter drive unit enables the hydraulic power from the drill rig to be transmitted to the cutter at the bottom of the self boring module and thus rotates the cutter while the cutting shoe is being pushed vertically by the drill rig.

A water or drilling mud pump is required for inserting the pressuremeters. The pump should have such capacity that it can reliably pump water with sand suspension or a slurried clay at a pressure of about 50 pounds per square inch and at a flow rate of about four gallons per minute. Either mono-type or piston pumps may be used.

The casing and the cutter drive rod are other pieces of major equipment needed apart from the probe and the drill rig. EX casing is available in 3 foot, 5 foot, or 10 foot lengths. The cutter drive rod is supplied by the manufacturer. It is essential that the necessary adapters and connections are compatible with both the drill rig as well as the casing and the drive rod.

The expansion test, which is the form most generally conducted with the self boring pressuremeter, can be performed either as a constant rate of strain or a constant rate of stress test. This probe permits conducting the tests at incremental rates of stress. The Cambridge In Situ Company

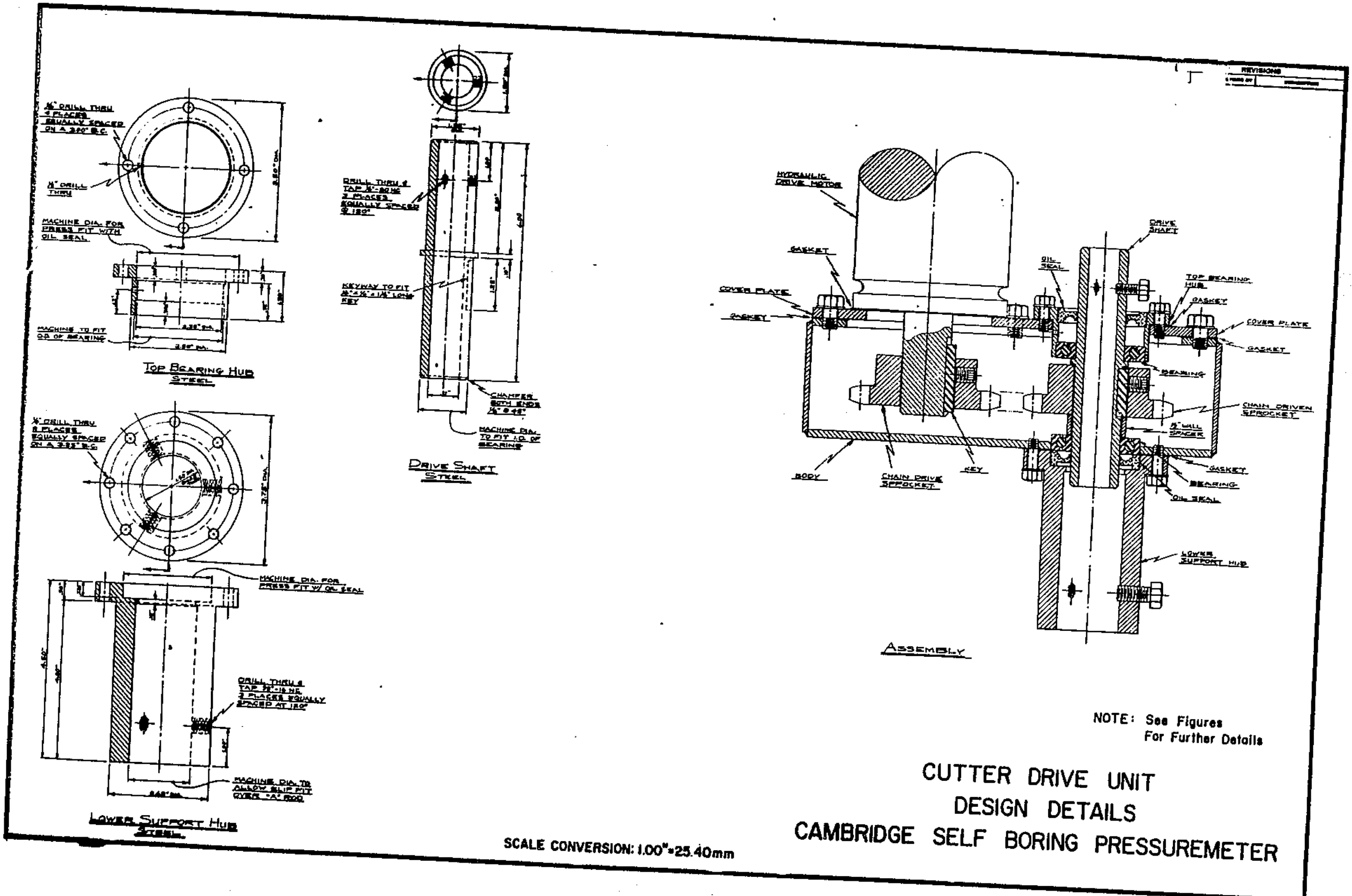


Figure 35 CUTTER DRIVE UNIT - DESIGN DETAILS - 1

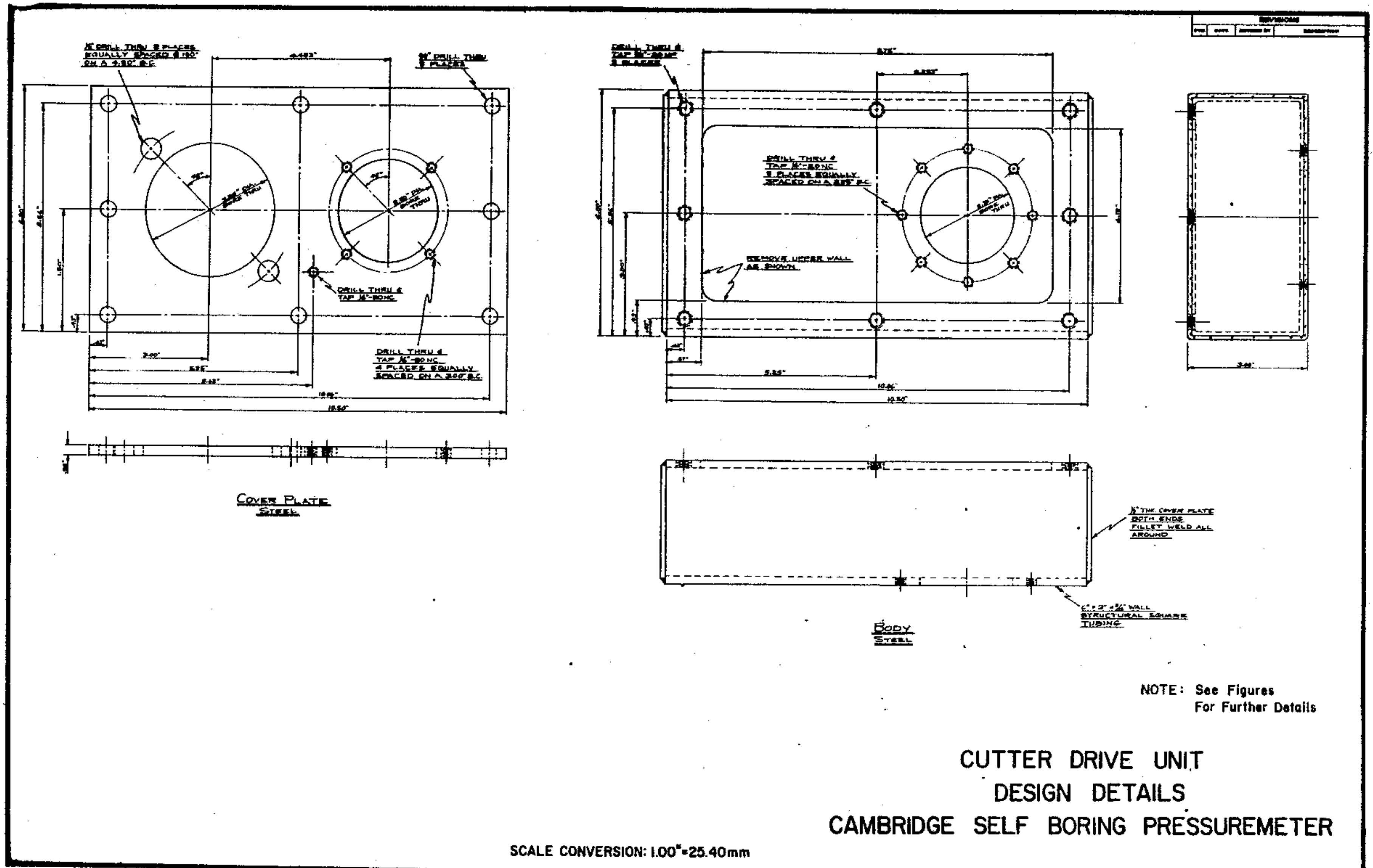


Figure 36 CUTTER DRIVE UNIT - DESIGN DETAILS - 2

supplies a strain-controlled expansion unit. The unit employs specially modified magnetic valves that operate in response to strain signals returning from the probe embedded in the ground. The strain rates possible with this unit range from 1 to 10 percent. Using this control unit, pressure measurements as well as total earth pressure measurements can be made.

To record and document the signals that come from the probe, a digital multimeter is required. This is a rugged water-tight electronic box which is well protected against electrical and mechanical misuse. It runs on normal vehicle electrical supply or from a separate 12-volt car battery.

An X-Y-Y plotter is used to follow the behavior of soil as the expansion test proceeds. The horizontal axis, or X axis, records the strain in percent of the deformation. The vertical (Y axis) records both total pressure as well as pore pressure changes. As the test proceeds, the shape of the pressuremeter curve will indicate soil disturbance.

At least two vehicles are required: one to transport the drill rig (Concore N-69) and the other to carry the probe, the accessories, and the water tank. The cylinder required to carry the gas under pressure is secured to one of the vehicles for safety.

The Cambridge Probe used in this evaluation is shown in Figure 37. The general test setup is shown in Figures 38A,B.

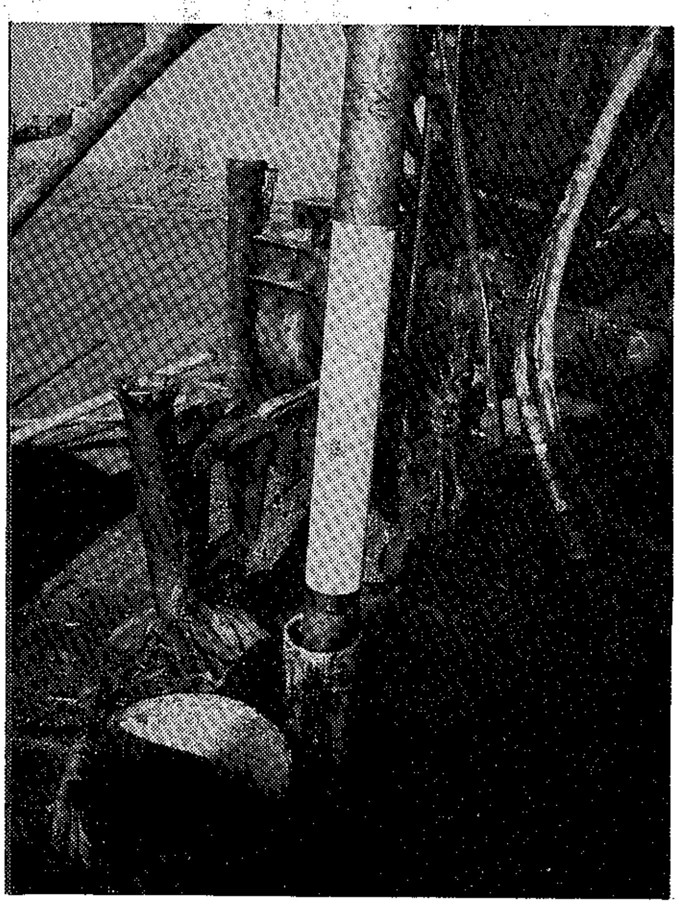


Figure 37 CAMBRIDGE SELF BORING PRESSUREMETER

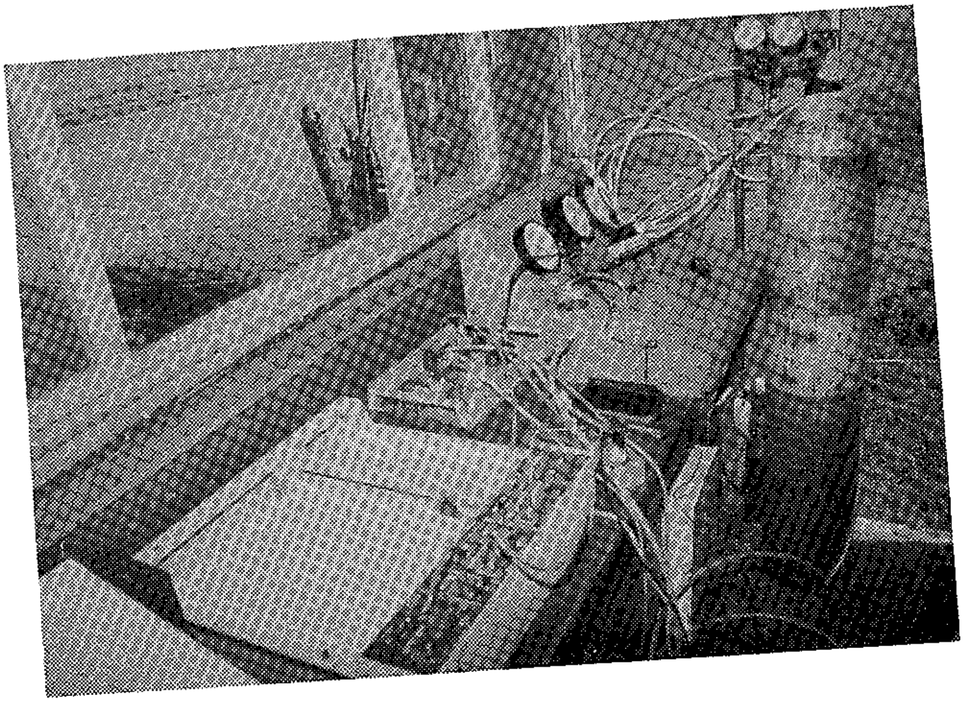
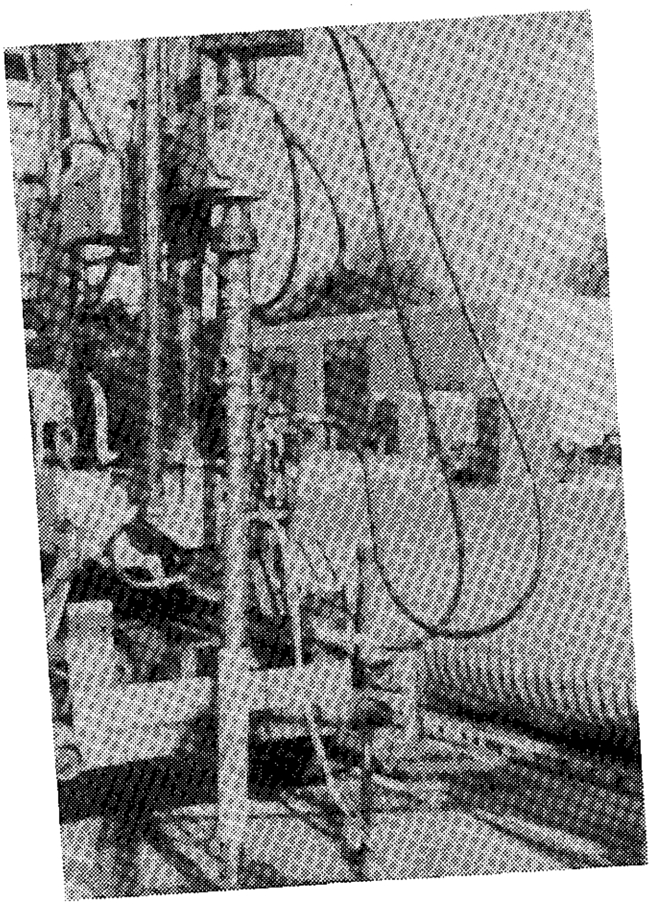
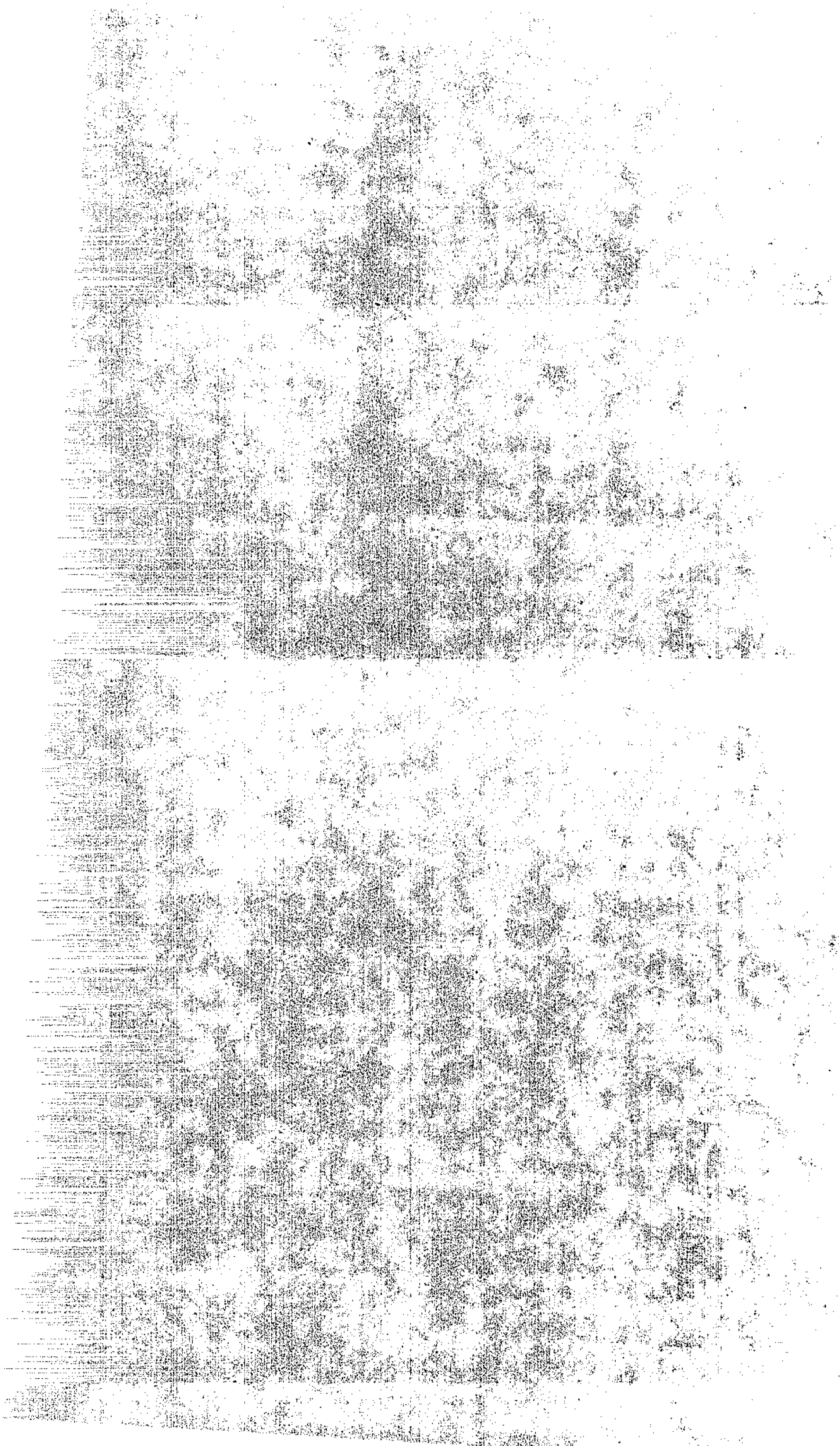


Figure 38 CAMBRIDGE PROBE TEST SETUP. (ABOVE) MONITORING SYSTEM, (BELOW) SELF BORING AND INSERTION SYSTEM





Modification and Adaptation

The manufacturer recommends the use of a "hydraulic ground frame" for insertion of the Cambridge self boring pressuremeter. However, TransLab used a light duty drill rig having the required hydraulic capability for this task.

The Concore Model N-69 drill rig is one of a series of Concore rigs used by Caltrans for subsurface investigation. It is trailer-mounted, with a hydraulic system that operates leveling outriggers and the down-feed mechanism. The rig was also equipped with a circulating pump.

The hydraulic system required slight modification to adapt the cutter drive unit (Figure 39) to the pressuremeter. This entailed plumbing-in valves to divert pressure to the drive unit and return.

For mounting the cutter drive unit, a section of "A" rod was used. The rod was inserted and clamped in the hydraulic chuck after which the cutter drive was slipped over the rod and clamped to it. An adapter for 1-1/2 inch galvanized pipe was welded to the bottom end of this "A" rod.

The outlet for return water was built from two short sections (6") of 1-1/2 inch galvanized pipe with a tee coupled between. Bronze bushings were brazed inside the galvanized pipe to stabilize the drive rod. Two pipe unions were incorporated for ease of removal. This assembly was threaded into the adapter at the end of the "A" rod.

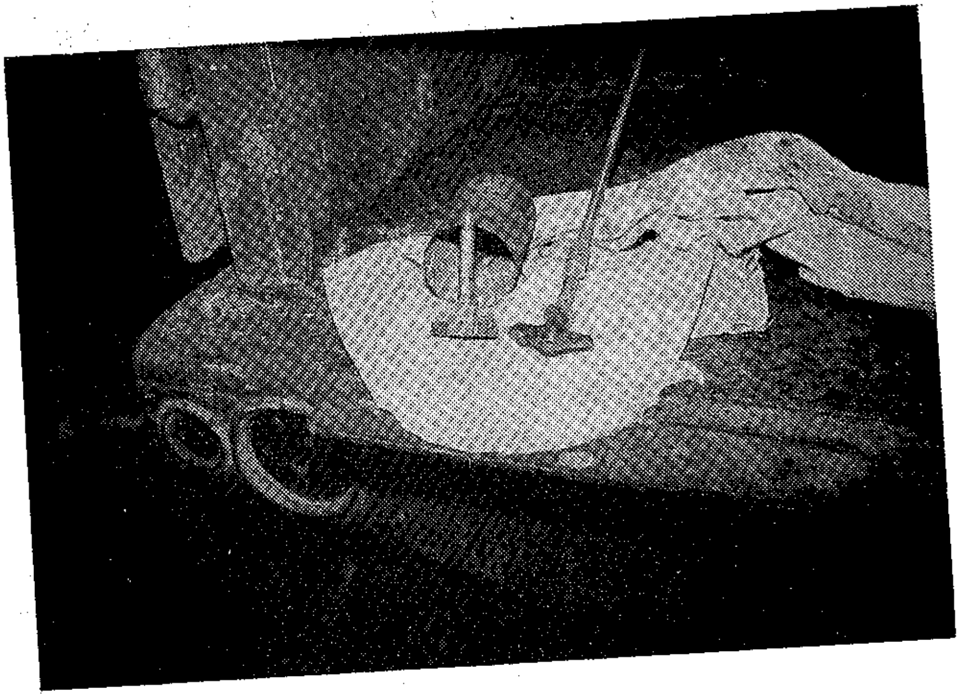


Figure 39 CALTRANS CUTTER DRIVE UNIT

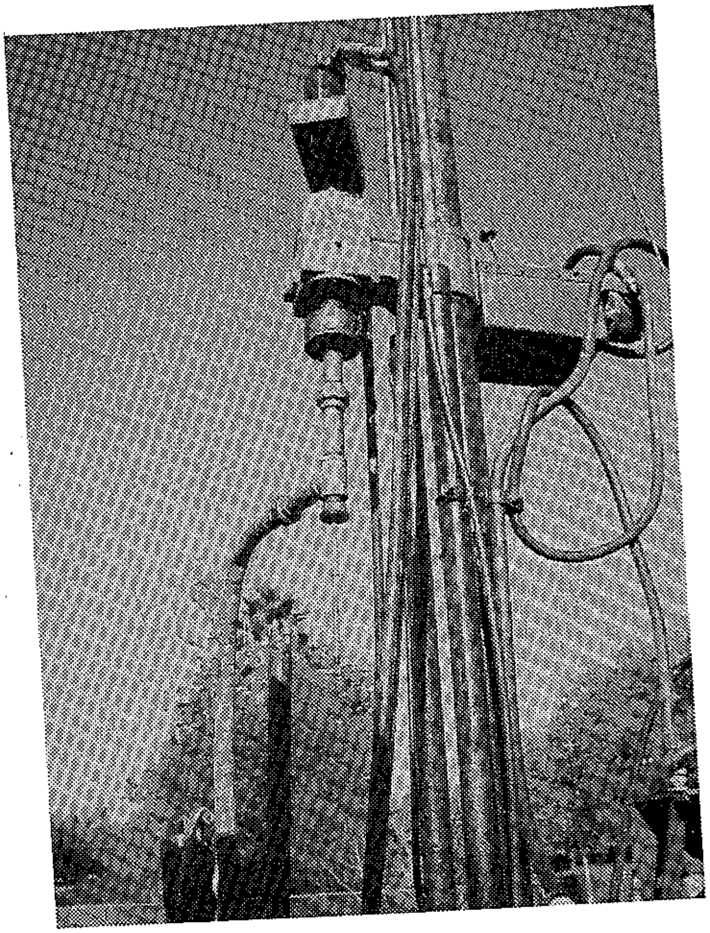


Figure 40 CALTRANS CUTTING SHOE

An adapter was made to enable coupling to the EX casing with 1-1/2 inch pipe threads at one end and EX thread at the other. With the EX casing adapter threaded to the bottom of the tee outlet assembly, addition of the 5-foot drive rod and casing sections was accomplished without necessitating the removal of the cutter drive unit. Water from the self-contained circulating pump was directed to the hollow drive rod via a rotating joint (water swivel).

After some relatively unsuccessful attempts at self boring, it was decided that part of the problem was the relative ease with which the bore of the cutting shoe became plugged. Consequently, a design for a new cutting shoe was developed (Figure 40) with a longer body and larger throat. The redesigned shoe was found to have less restriction to flow and improved cutting and mixing characteristics.

A weakness in the stainless steel drive rod coupler (at the cutting tool) which was causing failure under shock loading was corrected by replacement with one having an improved section modulus.

Operational Problems

The following were the major problems requiring solution before a meaningful self boring pressuremeter expansion or cyclic test could be conducted:

1. Not all components were available at the beginning of the project. Several had to be collected individually or designed and fabricated by the TransLab Machine Shop.

2. There were various problems with the electronic equipment and the plotter.

3. Shearing of the inner rods connecting to the cutting shoe was experienced.

4. Based on our experience in the field, new designs for the cutting shoe and the cutter were required.

5. A pressure-applying and monitoring system (console) had to be designed and fabricated.

An X-Y-Y plotter was borrowed from TransLab's electronic section for recording effective and total stress and strain readings in the form of plots. It was extremely sensitive to transient signals and noise. Extra grounding proved useless. The problem was solved through the use of a truck and camper, which were used as a field laboratory. Thus, wind and dust were kept from the unit.

At Site 1, the upper soil layers (4 meters in depth) contained angular material which quickly tore the membranes. Casing through the material and use of drill mud to help remove deleterious particles from the bottom of the cased hole allowed for success at lower depths where soil was more homogeneous.

Another problem became evident at Site 2 when boring at extended depths (beyond 10 meters). The membrane had a tendency to pull out of its lower retainer due to adhesion of the clayey soils. It was suggested by the supplier of the probe that a vacuum be applied within the membrane sucking the thin rubber up tight against the probe. This method was partially successful, although it changed initial conditions in the probe and raised some questions as to the accuracy of the initial readings.

Limitations and Advantages

The limitations and advantages have been very well summarized in Reference 68 and are here quoted.

I. "The instrument will not penetrate gravel, boulder clay, claystone, or similar materials.

II. "As in other forms of test the orientation of failure planes and the mode of deformation will usually be inappropriate to the field situation.

III. "There is no control of the total or of the effective stress path. In practice only two stress paths can be followed, one corresponding to an undrained test and the other corresponding to a fully drained test.

IV. "To reduce drainage effects undrained tests have to be performed at high rates of strain. This leads to undesirable rate effects which introduce errors into the analysis of the data. These effects, however, have been shown to be small.

V. "The instrument is complex by present day standards. However, much experience has now been accumulated both with the original equipment and with the commercial versions of it and this has shown the reliability of the equipment. It is also clear that recent developments in soil mechanics will require more sophisticated techniques in the future."

The advantages have been summarized also from Reference 68 as follows:

I. "The tests are performed on virtually undisturbed soil. Although some slight disturbance is inevitable it will be very greatly less than the disturbance associated with so-called "undisturbed sampling" or with a pressuremeter testing in a prebored hole.

II. "It is possible to obtain a number of soil parameters from one test. For clays, these parameters are the undrained shear strength, the shear modulus or the undrained Young's modulus and the in situ horizontal total stress. For sands, the angle of internal friction and the angle of dilation can be determined.

III. "These parameters can be derived from the test results using well developed theories of cavity expansion without resort to any empirical correction factors whatever.

IV. "Many of the variables associated with other methods of testing, such as the amount of disturbance due to sampling or to trimming, the time between finishing the borehole and starting to test or take samples, the effects of piping a borehole in sand or soft chalk below the water table, and so on, are all eliminated. As a consequence, these tests produce very much less scattered data than other methods.

V. "It is possible to obtain results very quickly when necessary as the test data can be processed on site with the aid only of a pocket calculator or slide rule, a pencil, ruler, and graph paper. No laboratory testing is required."

Applications to Geotechnical Problems

There are many applications of data resulting from expansion tests and cyclic tests. The geotechnical applications of self boring pressuremeters test data have been documented in Chapter 7 of Reference 2. They can be summarized as follows:

1. Identification of soil type.
2. Calculation of bearing capacity of foundations.
3. Settlement estimates of shallow foundations.
4. Prediction of the lateral reaction of piles.
5. Estimation of shear strength.

Examples of applications are presented in Reference 2.

CHAPTER 9. TEST RESULTS AND DISCUSSION

The field data from each of the five probes are presented below in the same order in which these probes were described in the previous chapters; namely, Borehole Shear Device, Vane Shear Device, Dutch Cone Penetrometer, Standard Penetrometer, and the Cambridge Probe.

Borehole Shear Device

Typical envelopes are presented in Figures 41 and 42. They are developed by plotting the normal stress in pounds per square inch on the X axis and the shear stress in pounds per square inch on the Y axis. Usually, three tests were conducted in the initial position in which the two shear blades gripped the sides of the hole. The two shear plates were then rotated 90° and the test conducted in the perpendicular position. The tests were repeated three times in this new position and the results plotted. It can be observed in general, from these seven plots that the envelope for each soil tested at a particular depth has almost the same slope (Figures 41 and 42). From these typical envelopes, the cohesion and the friction angle values were determined. The shear strength values in tons per square foot could be calculated from the known values of cohesion and friction angle and the depth at which the test was conducted.

Shear strength-depth profiles are presented in Figures 43 through 45. At all sites, the tests conducted were at depths between one and two meters. The laboratory tests were conducted on samples from much lower depths, so no correlations

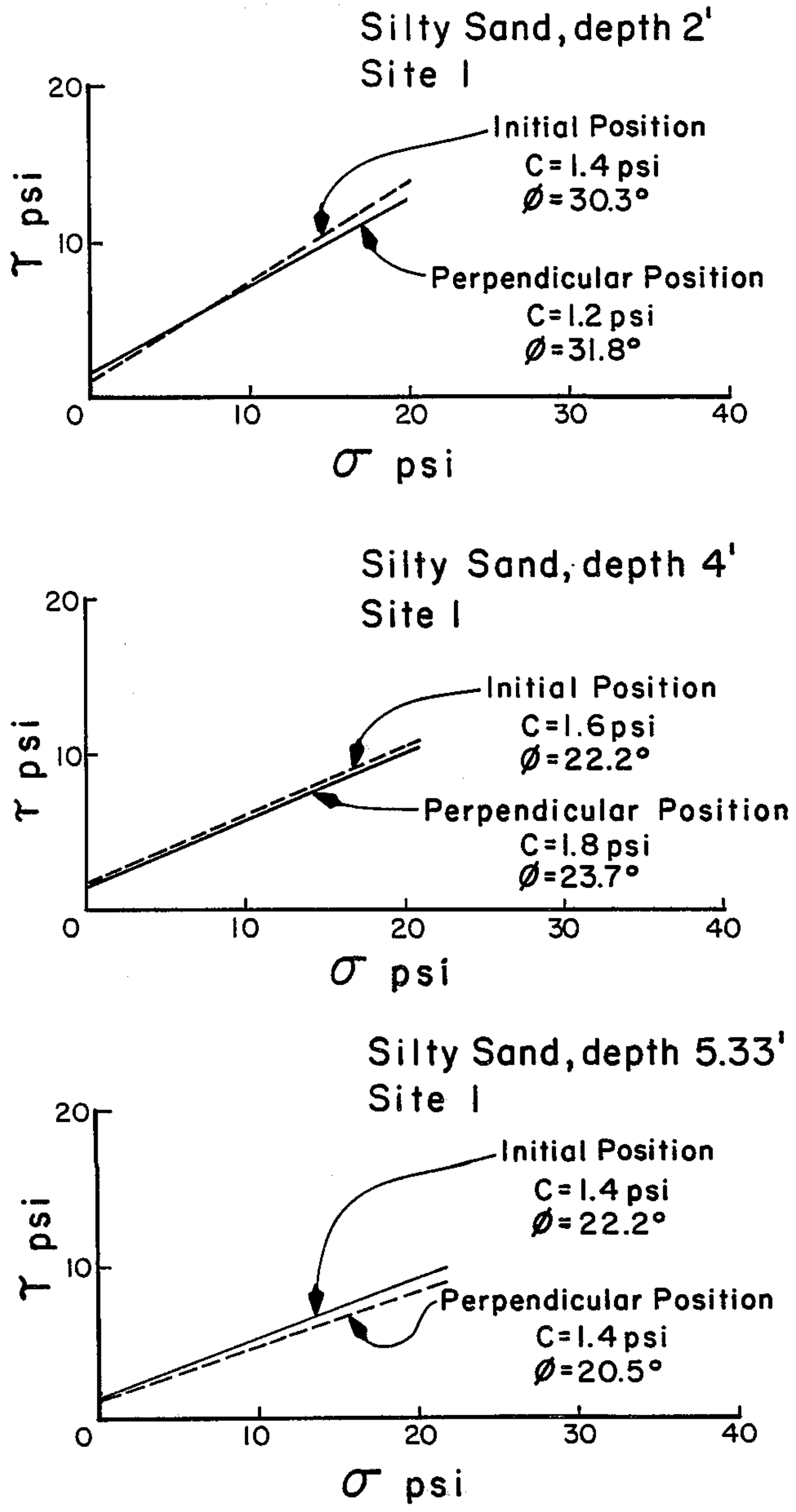


Figure 41 ENVELOPES FROM BOREHOLE SHEAR TEST, SITE 1

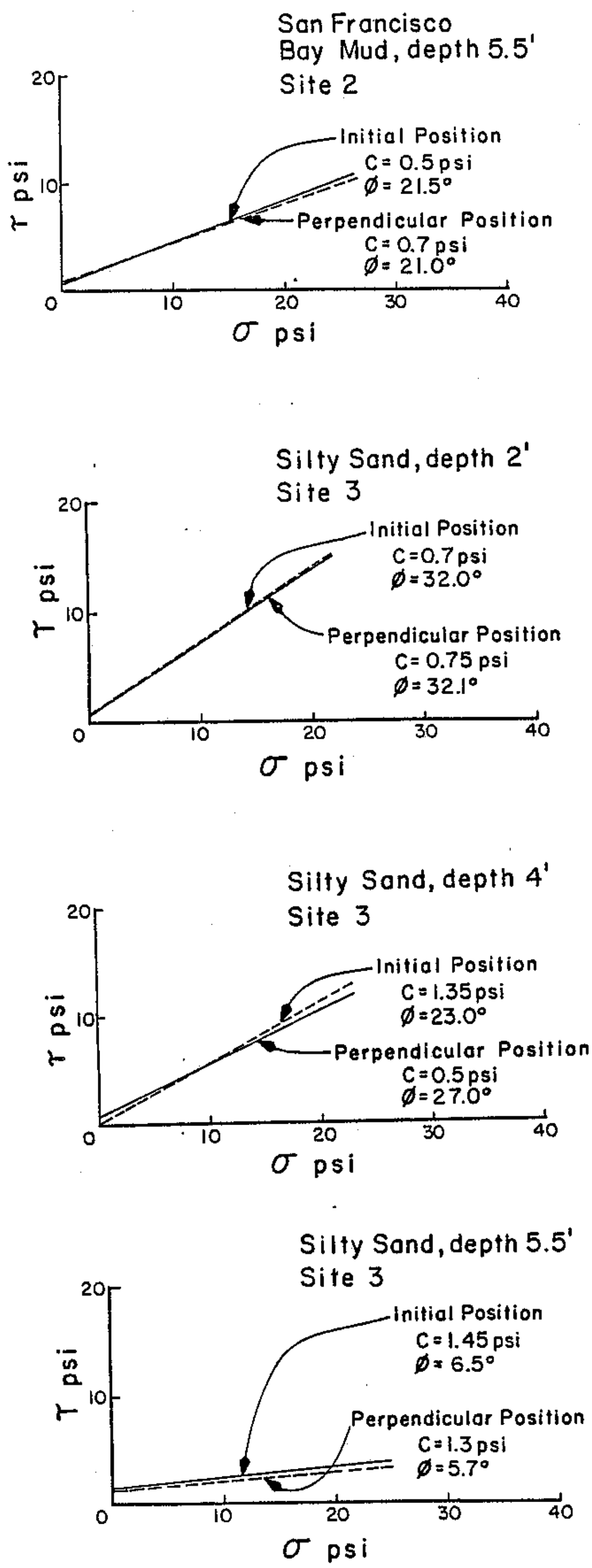


Figure 42 ENVELOPES FROM BOREHOLE SHEAR TEST, SITES 2 AND 3

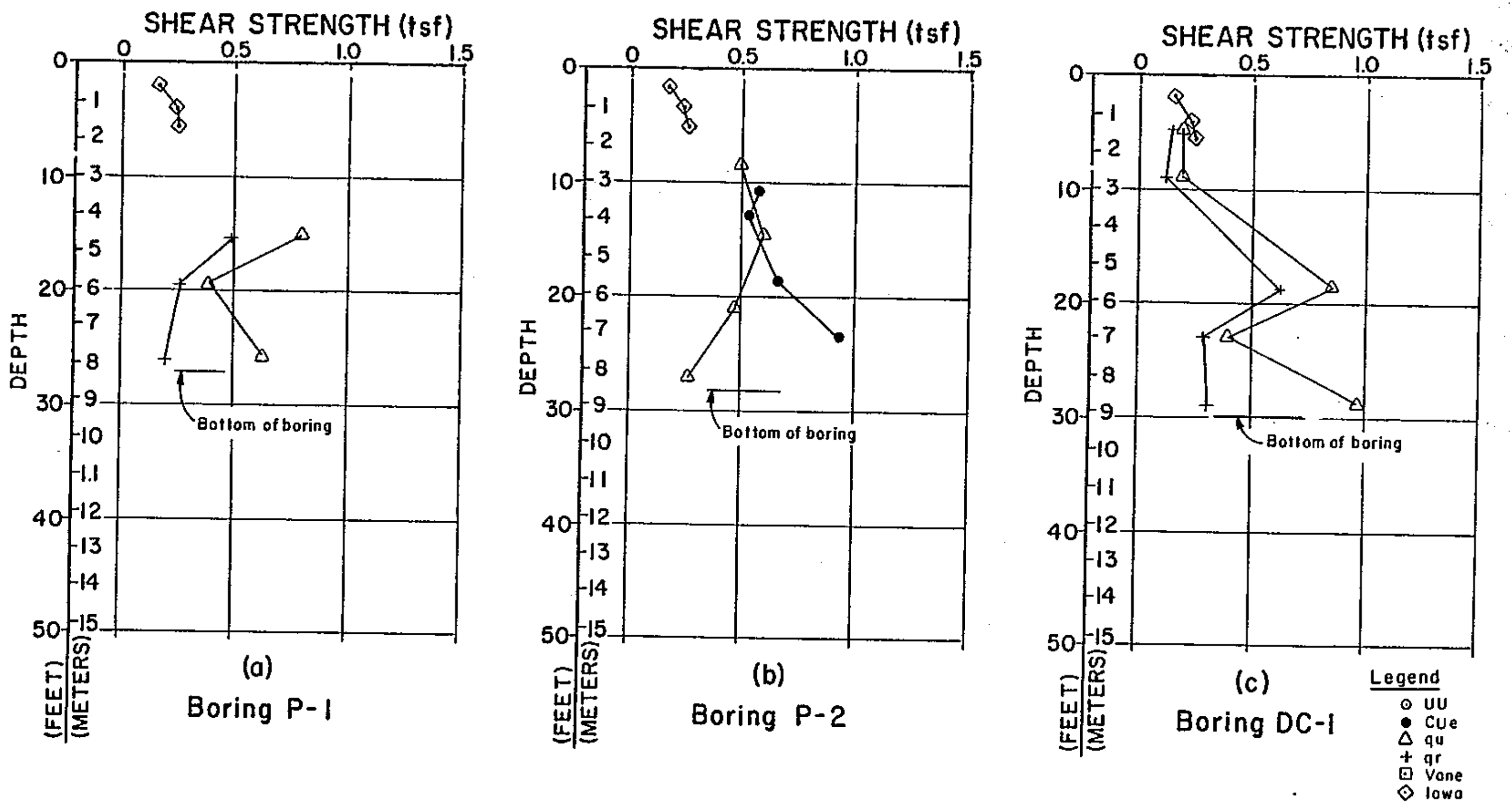


Figure 43 SHEAR STRENGTH CORRELATION - IOWA, SITE 1

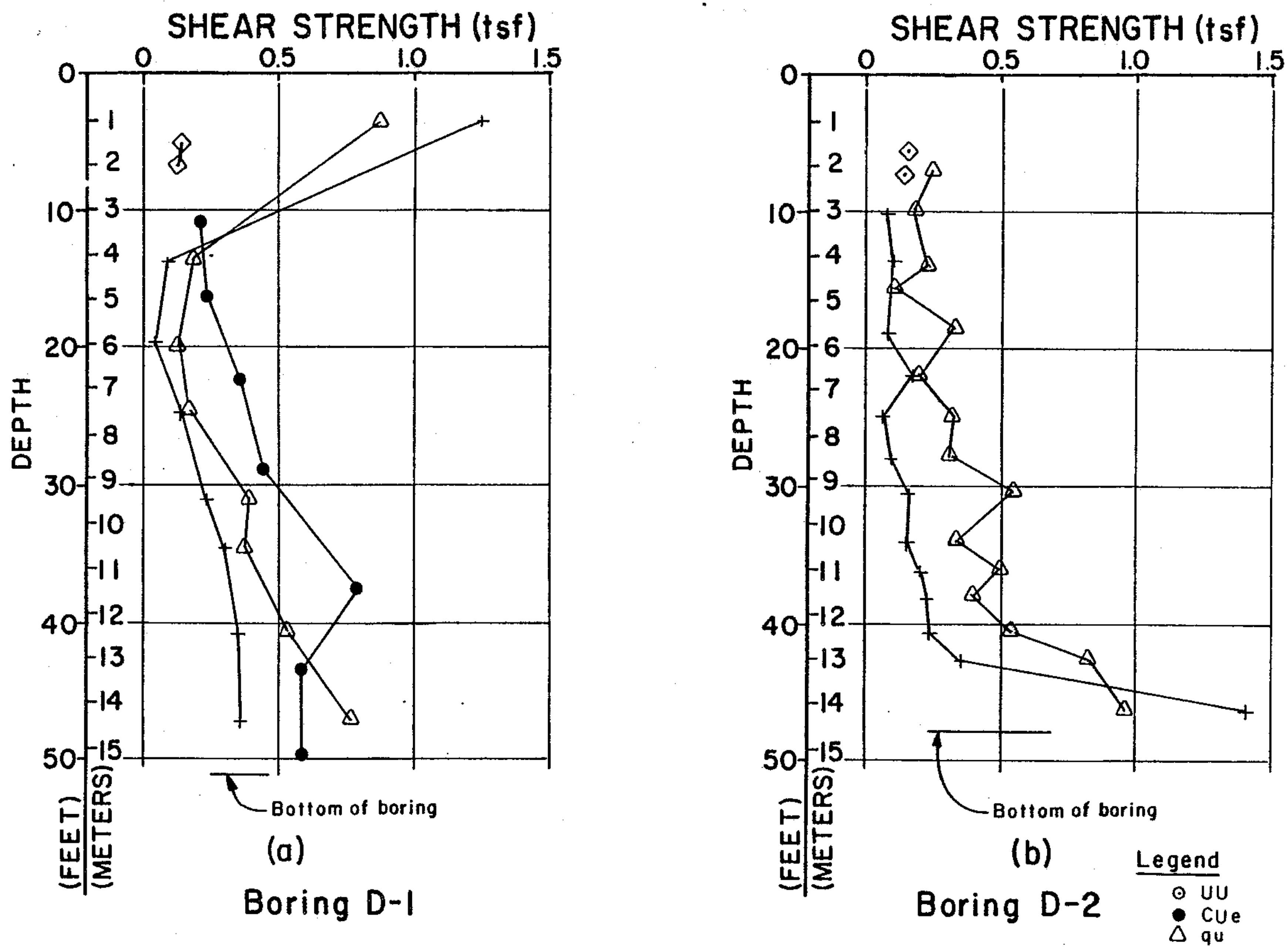


Figure 44 SHEAR STRENGTH CORRELATION - IOWA, SITE 2

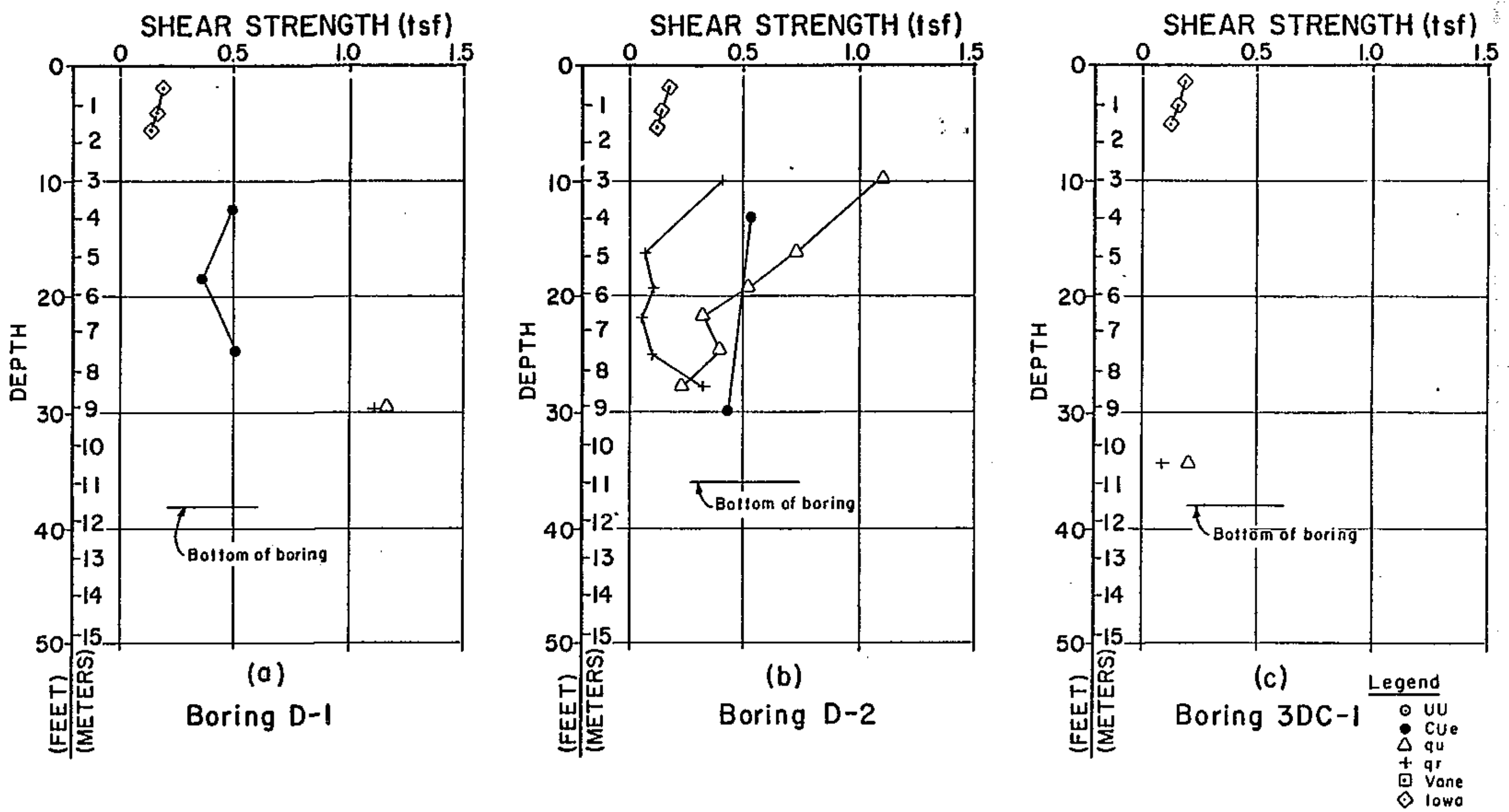


Figure 45 SHEAR STRENGTH CORRELATION - IOWA, SITE 3

or comparisons could be made. In Boring DC-1 (Site 1) there were two tests at 1-1/2 meters depth that produced results very close to those from laboratory unconfined shear strength tests. Tests could not generally be conducted at comparable depths because of the caving experienced in the holes. Hence, use of this probe is limited primarily to borings in which caving does not occur.

Vane Shear Device

The vane test data are presented in Figures 46, 47 and 48. At Site 1, the shear strength obtained by using the vane in the field is found to be between the unconfined compressive strength of soil in undisturbed and remolded condition. In all cases, the vane shear strength was less than that obtained by using consolidated undrained shear strength with pore pressure measurements. It was found that the shear strengths obtained using the vane in the field were very close to the remolded shear strength from the unconfined compressive strength test. In all cases, vane shear strengths were the lowest measured both from the laboratory tests and from other in situ devices.

At Site 3, which consisted mostly of peat and peaty soils, it was found that the vane gave strength values which were between unconfined compressive tests in the undisturbed state and the remolded state. These data are presented in Figure 19.

Dutch Cone Penetrometer

The field data obtained using the Dutch Cone Penetrometer, are presented in Figures 49 and 50. Each of them contains

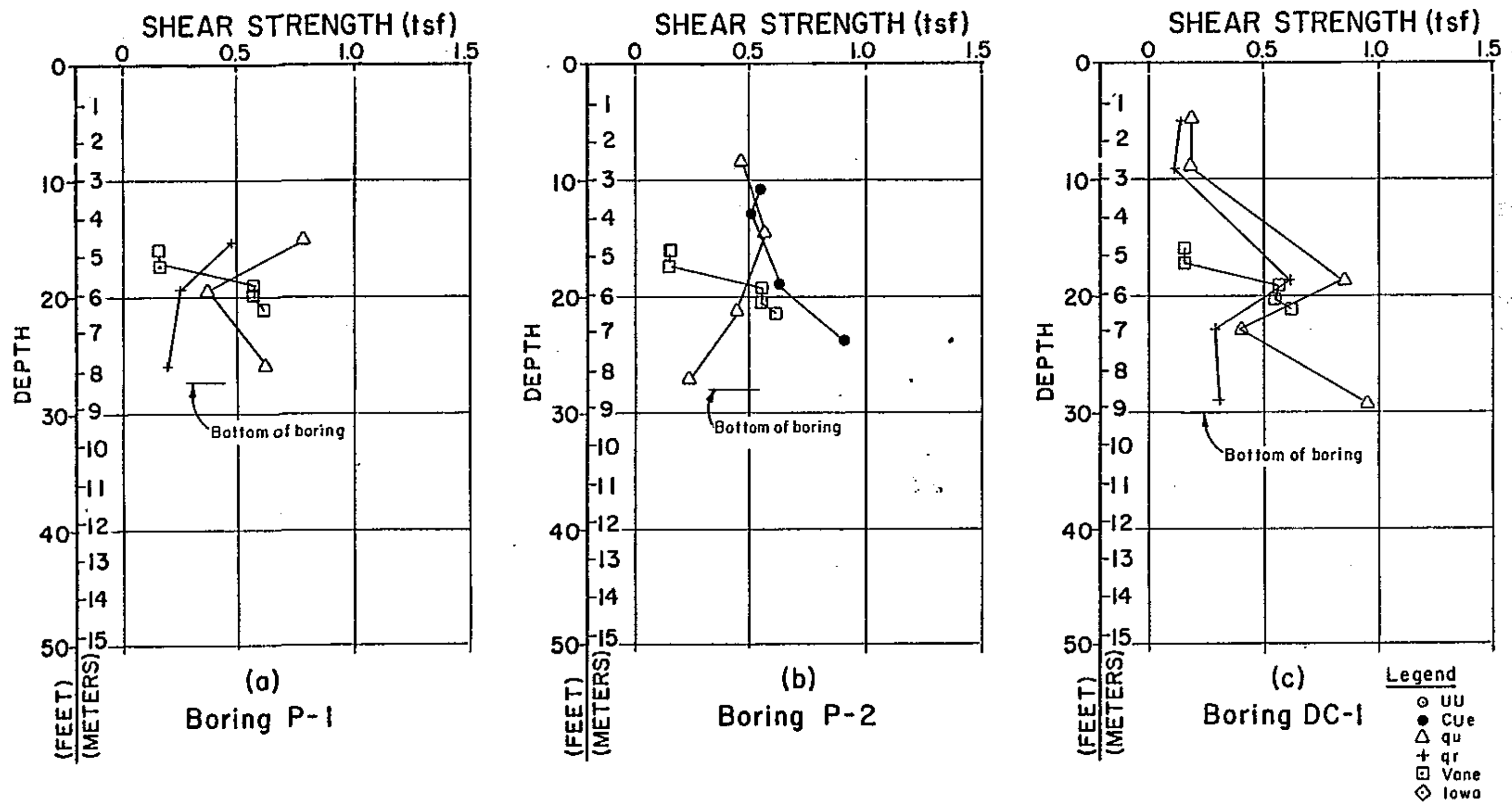


Figure 46 SHEAR STRENGTH CORRELATION - VANE, SITE 1

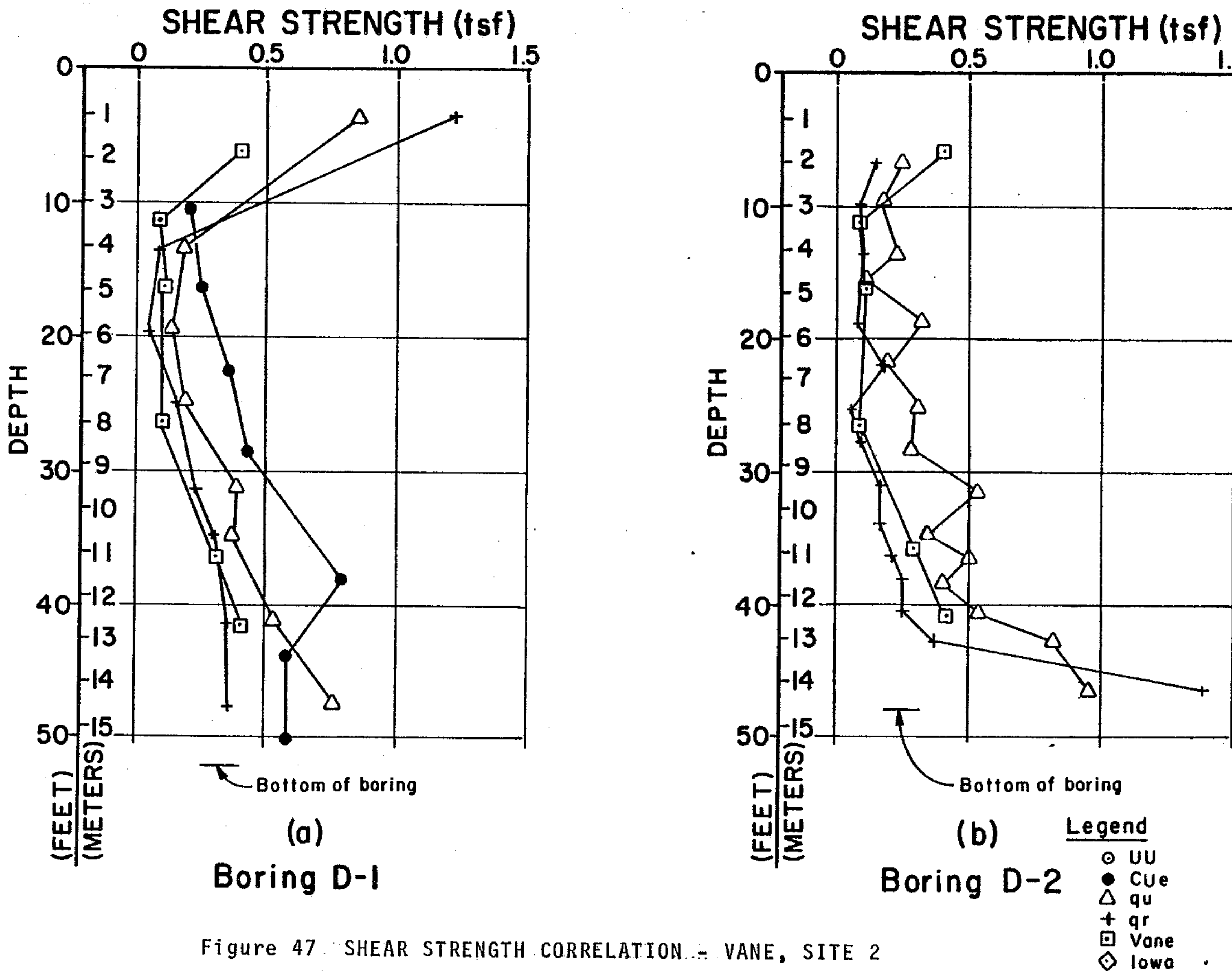


Figure 47 SHEAR STRENGTH CORRELATION - VANE, SITE 2

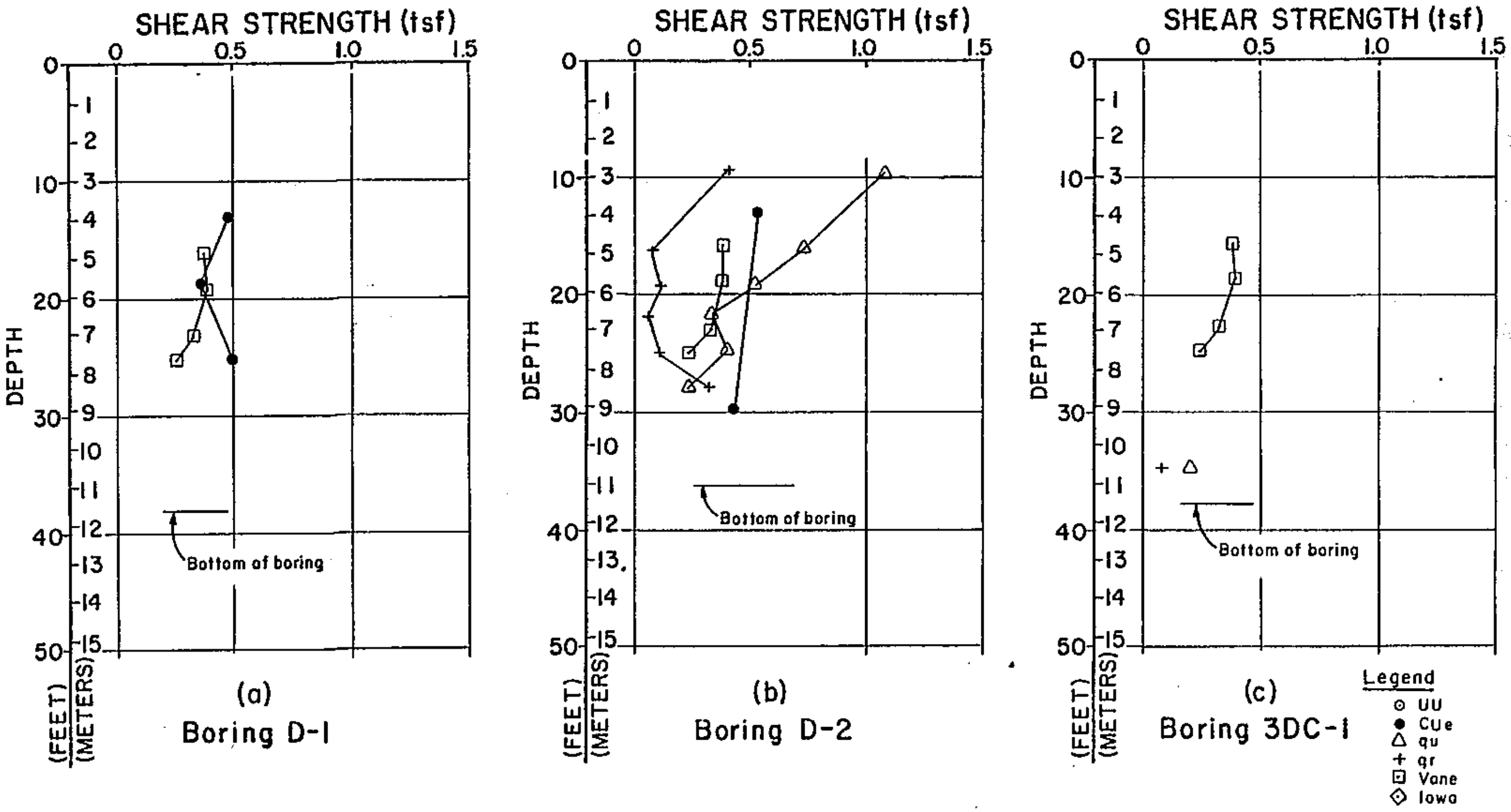


Figure 48 SHEAR STRENGTH CORRELATION - VANE, SITE 3

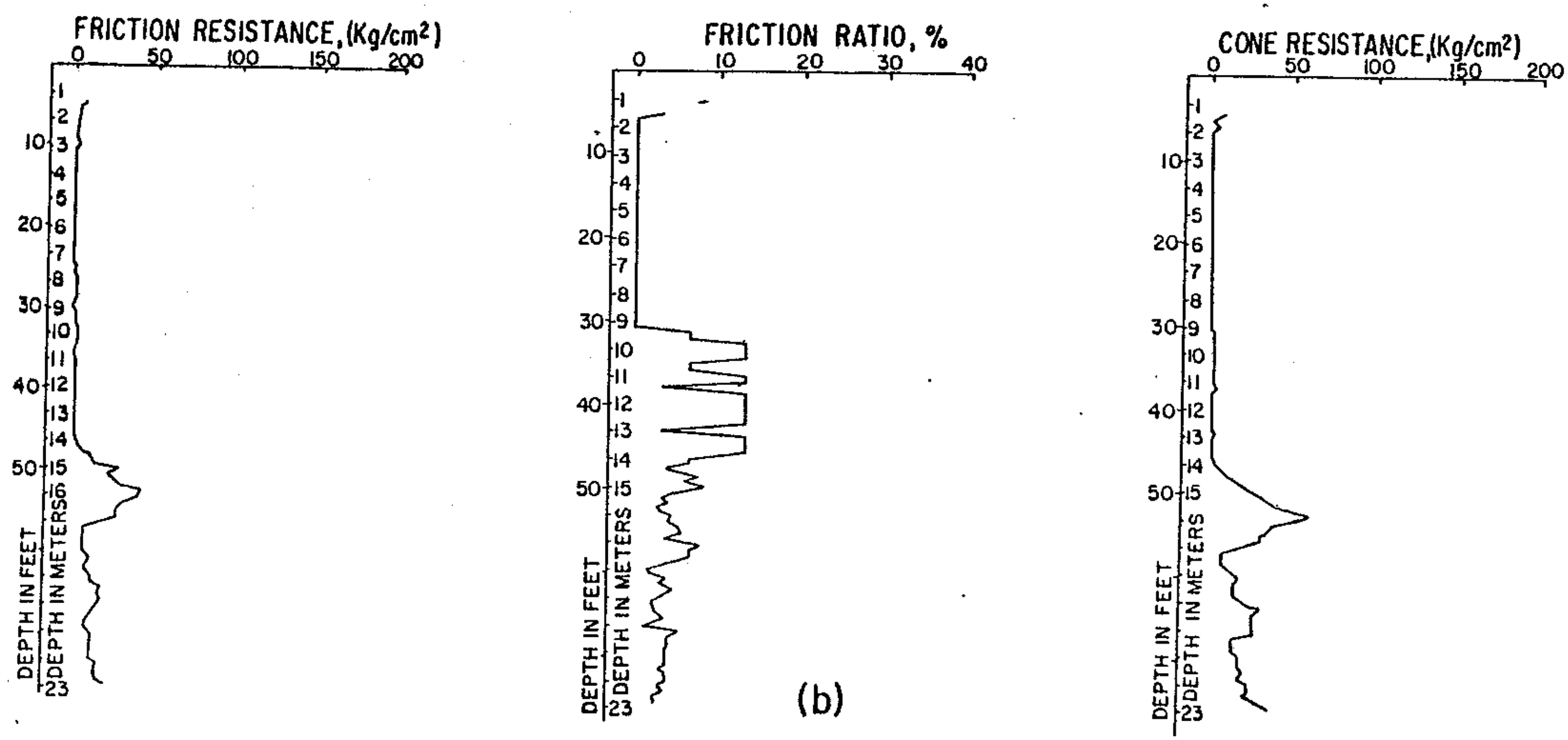


Figure 49 DEPTH PROFILES OF DUTCH CONE TEST DATA, SITE 2

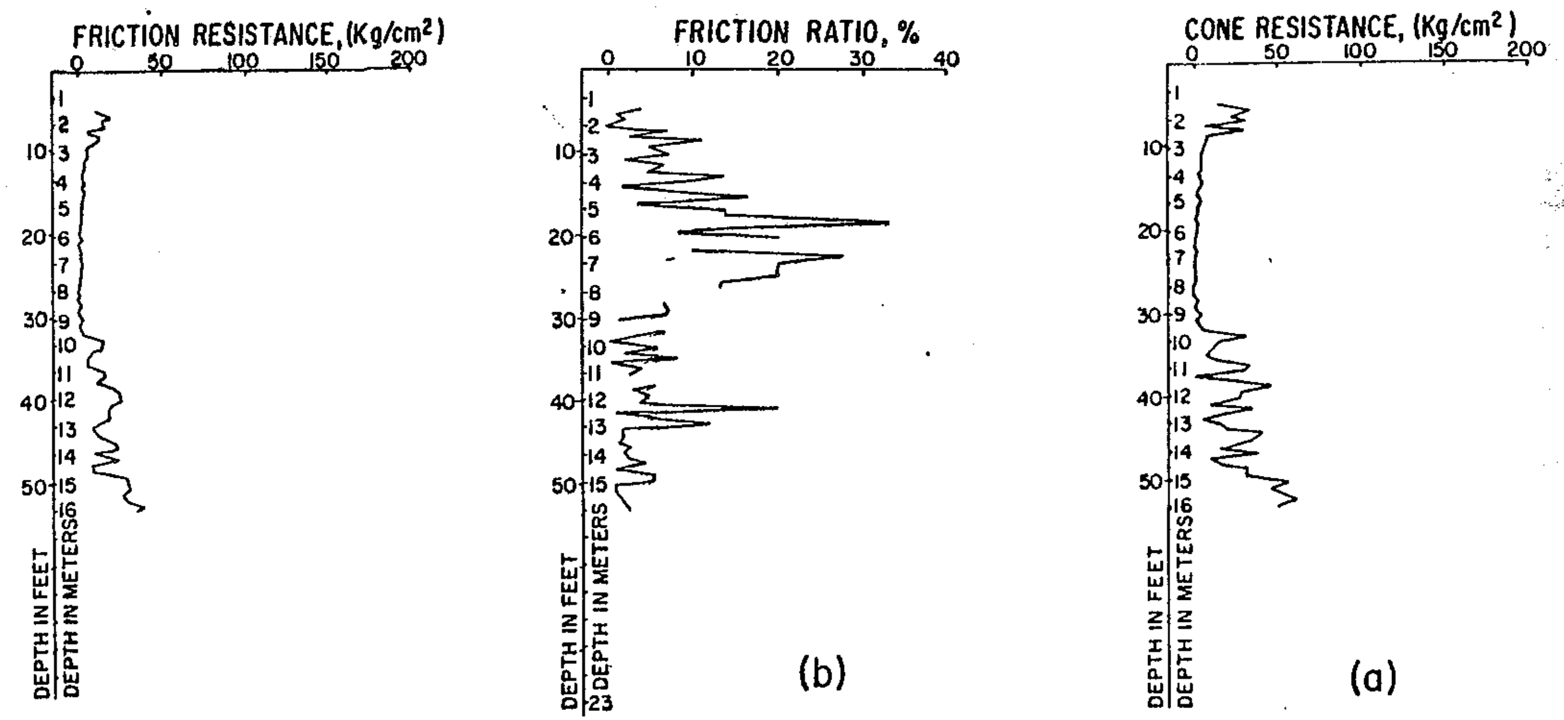


Figure 50 DEPTH PROFILES OF DUTCH CONE TEST DATA, SITE 3

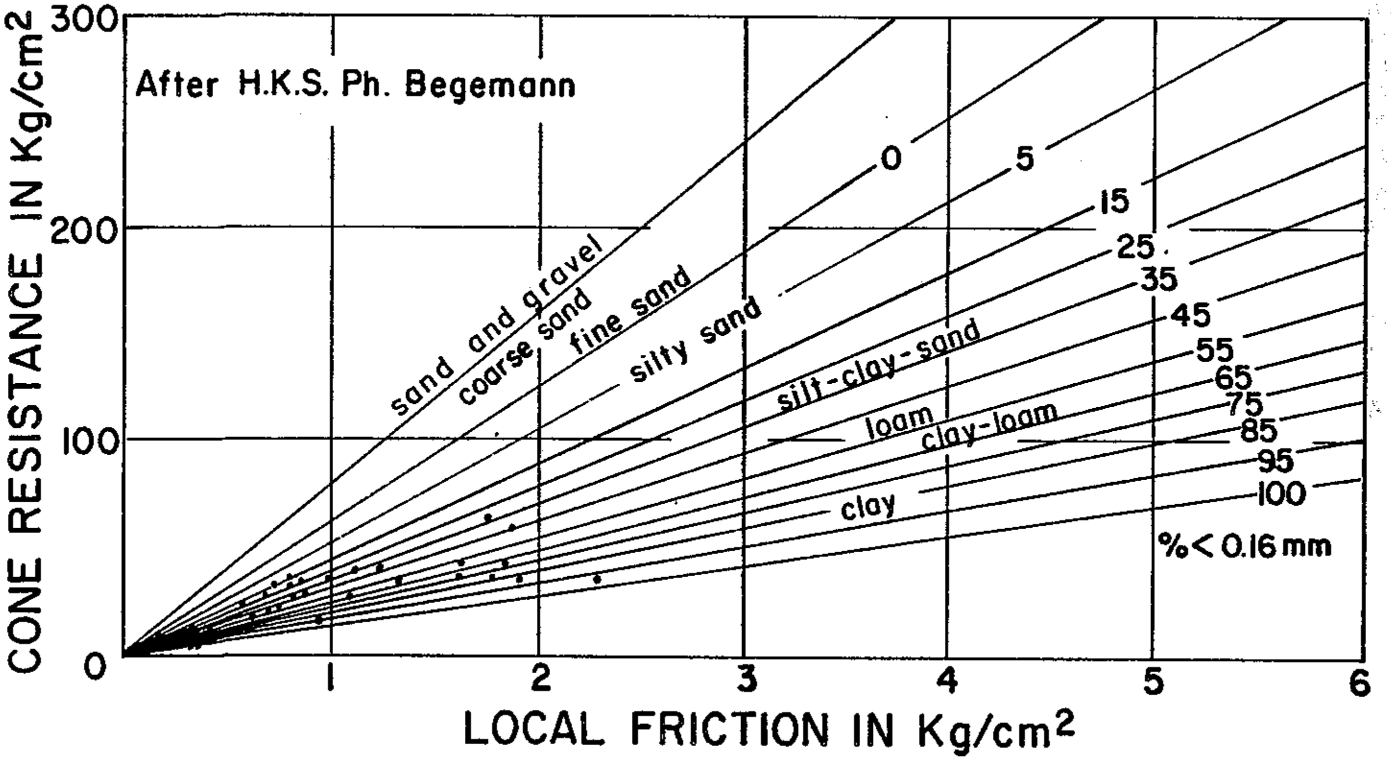
three depth profiles showing the variation with respect to depth of cone resistance, friction ratio, and friction resistance. These depth profiles can be used for developing subsurface information as explained below.

Soil type can be determined using two kinds of charts. The first was developed by Begemann (Figure 51). In it, two parameters - local friction and cone resistance - are related by a series of radiating lines from the origin. This permits soil classification from the clay to gravel range. The various soils identified in Sites 1, 2, and 3 have been presented in Figures 53 through 56.

In Schmertmann's chart (Figure 52) for soil identification, friction ratio and cone resistance are related and are further subdivided into various soil types. Application of the chart to the soils encountered in this study is shown by Figures 53 through 55. A comparison of the soil types predicted using the Schmertmann and Begemann chart descriptions from boring logs was made for all three sites. These comparisons are also presented in Figures 53 through 55.

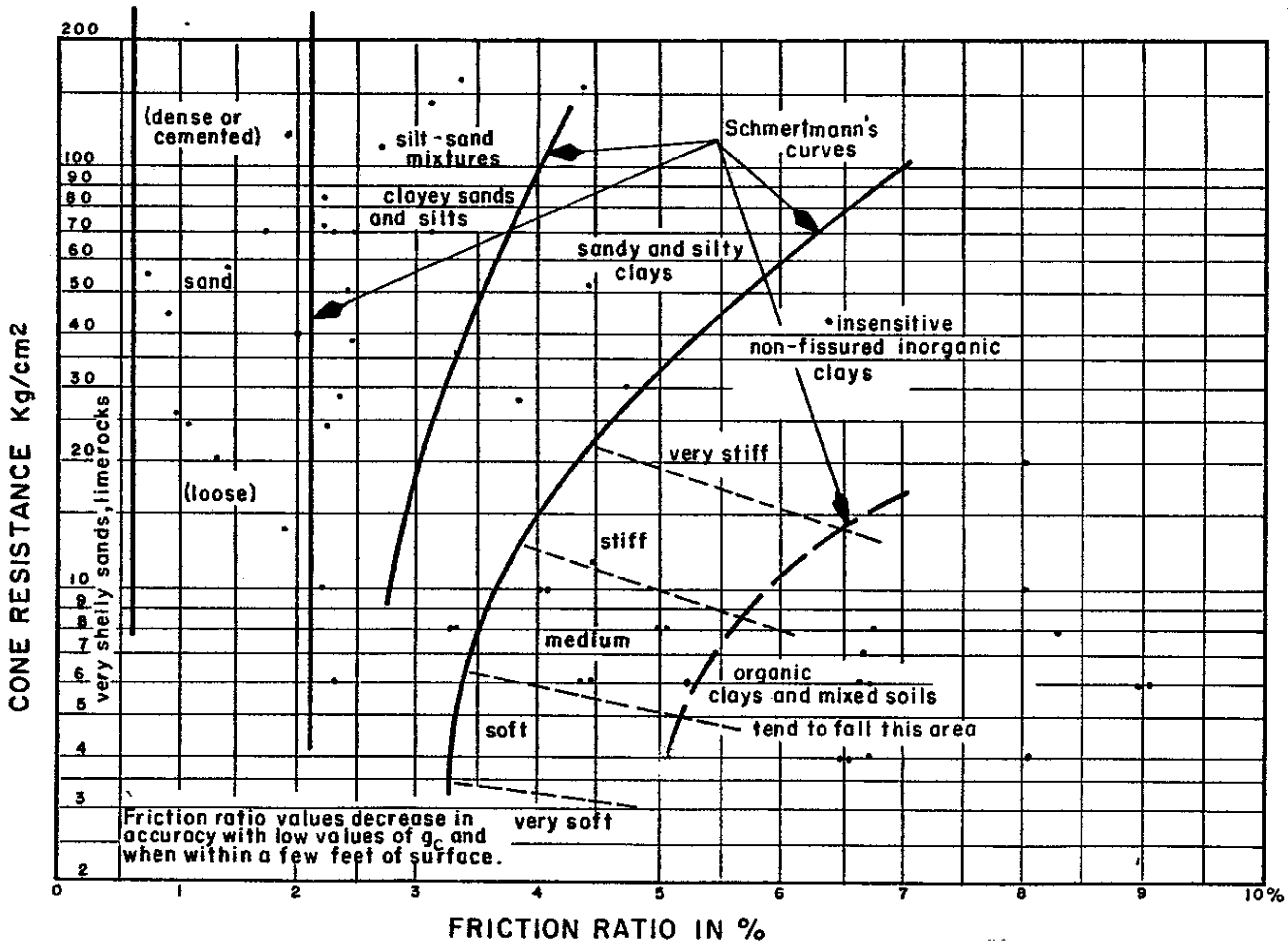
The Dutch Cone data can be used to predict undrained shear strength of soil. Many investigators have proposed methods and factors to be applied in estimating the shear strength of soil from Dutch Cone data. Begemann(69) has suggested a value of 13.6 to be used for this purpose as compared to 16 suggested by Meigh and Corbett(70).

Both factors were used in this study for estimating the shear strength from all sites. At each site, the Dutch Cone tests were conducted using the continuous as well as



From Ref 26

Figure 51 SOIL IDENTIFICATION USING BEGEMANN'S CHART



From Ref 27

Figure 52 SOIL IDENTIFICATION USING SCHMERTMANN'S CHART

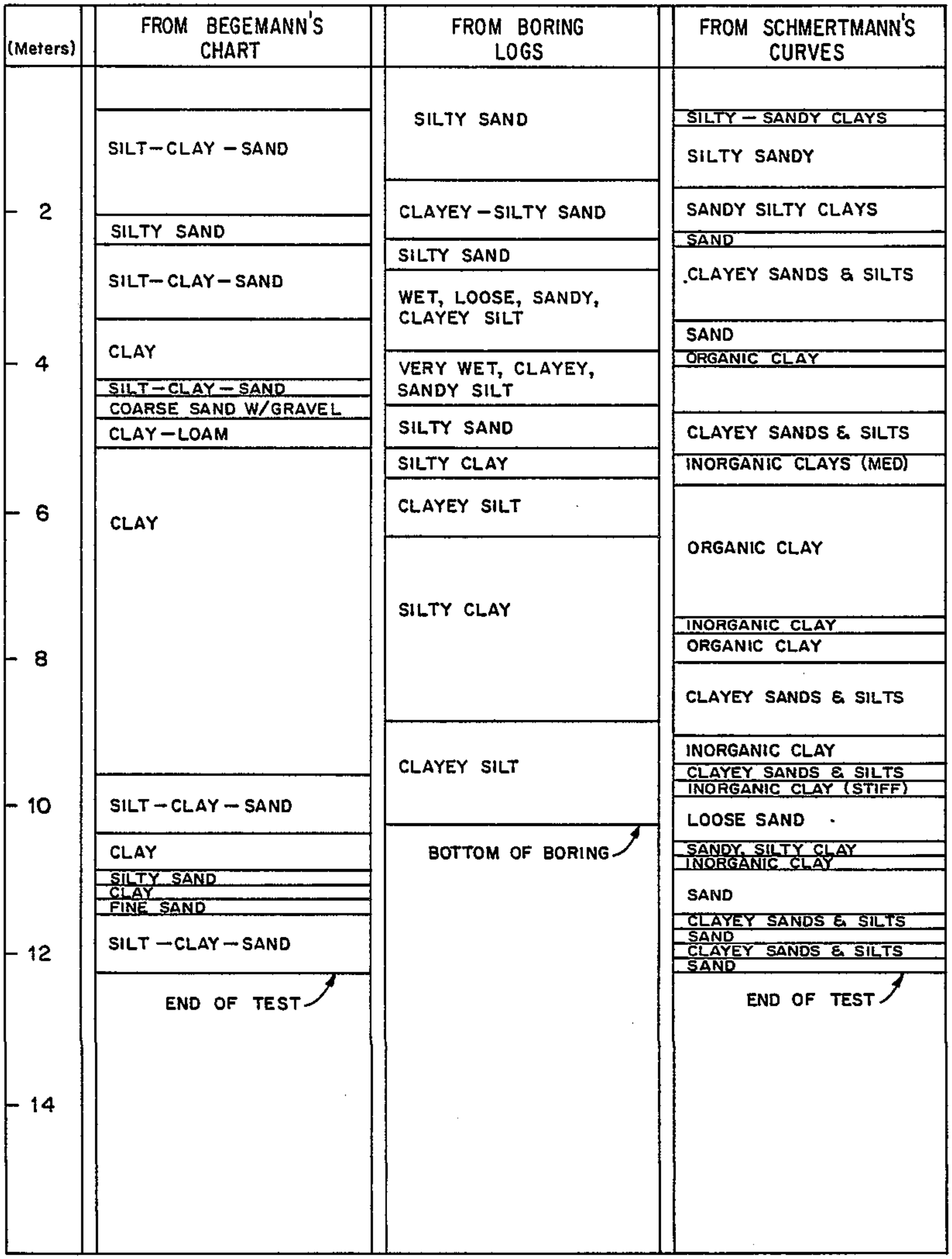


Figure 53 COMPARATIVE STUDY OF SOIL IDENTIFICATION, SITE 1

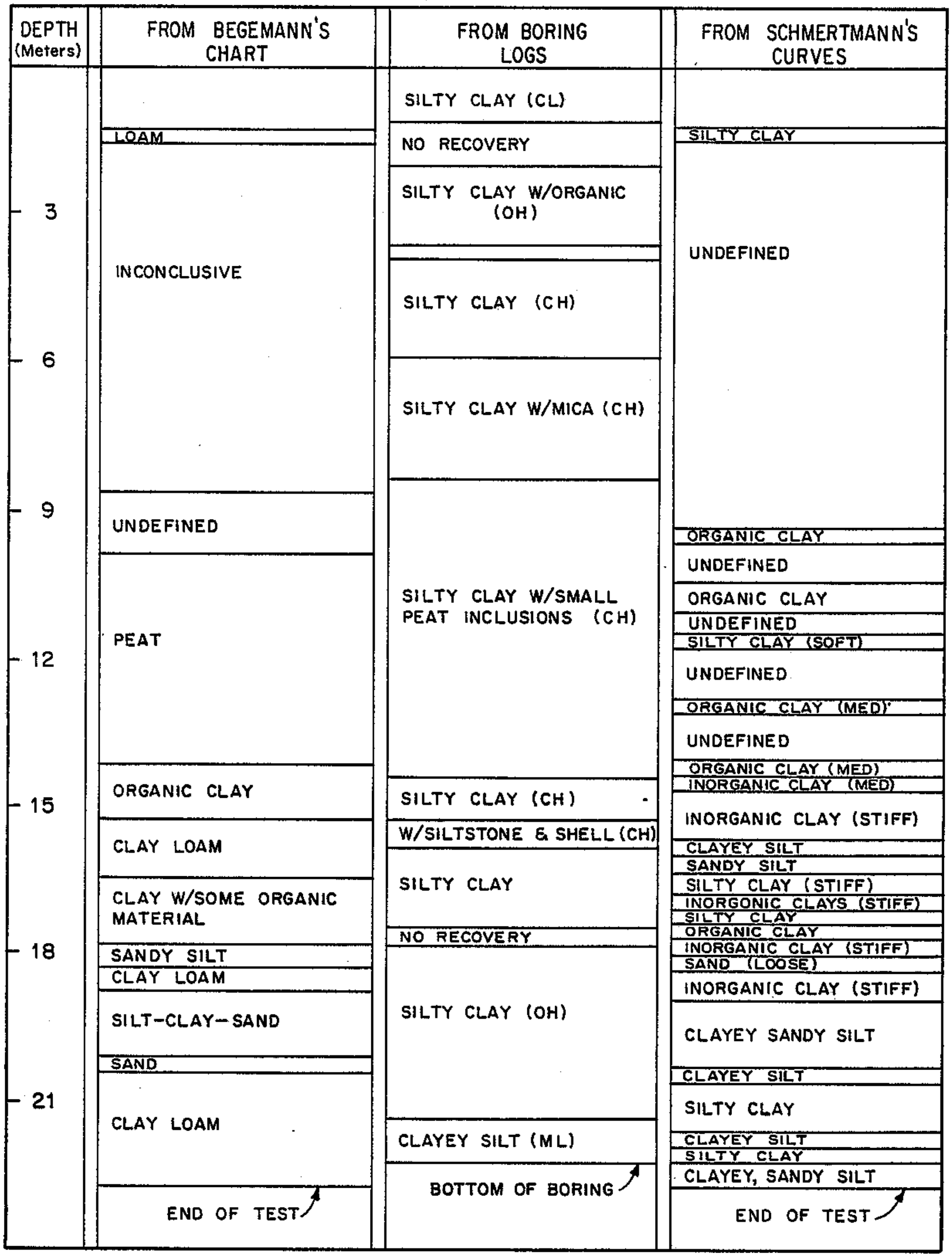


Figure 54 COMPARATIVE STUDY OF SOIL IDENTIFICATION, SITE 2

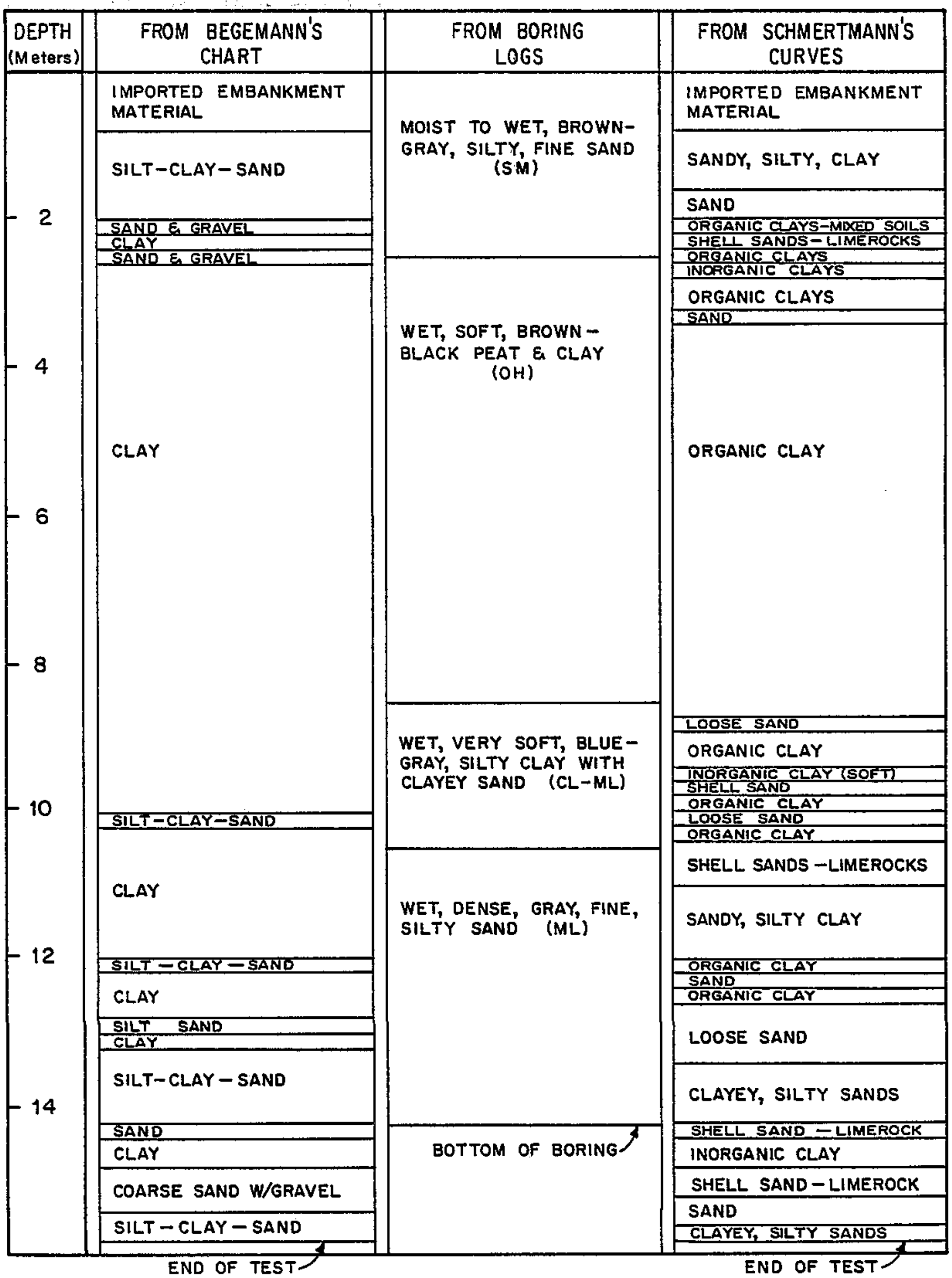


Figure 55 COMPARATIVE STUDY OF SOIL IDENTIFICATION, SITE 3

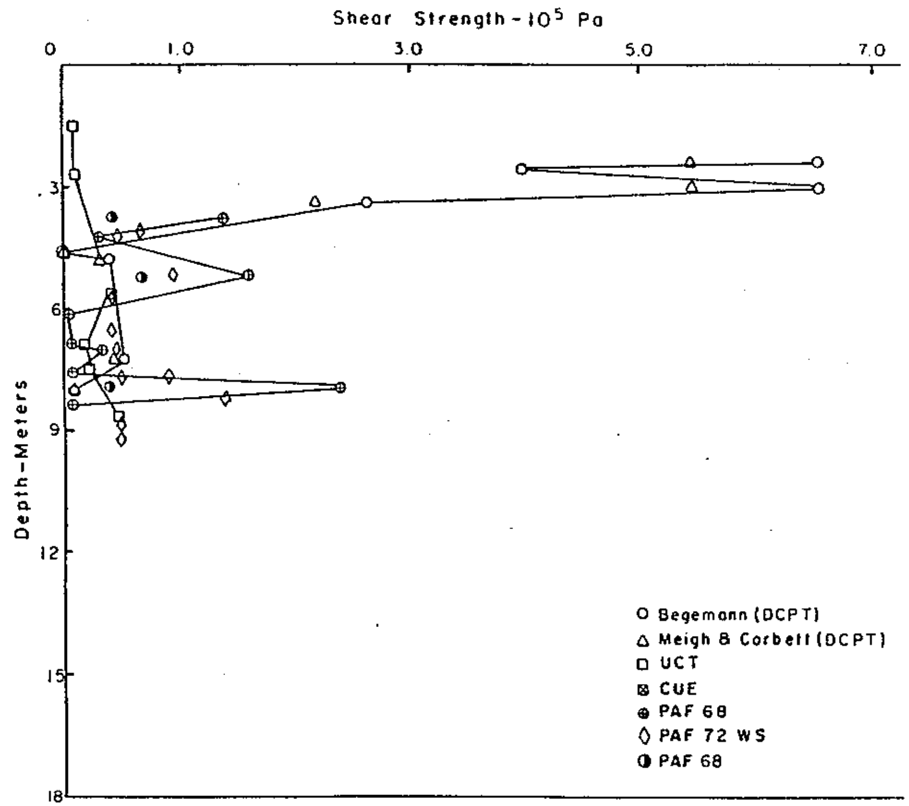


Figure 56 COMPARATIVE STUDY OF SHEAR STRENGTH, (CONTINUOUS), SITE 1

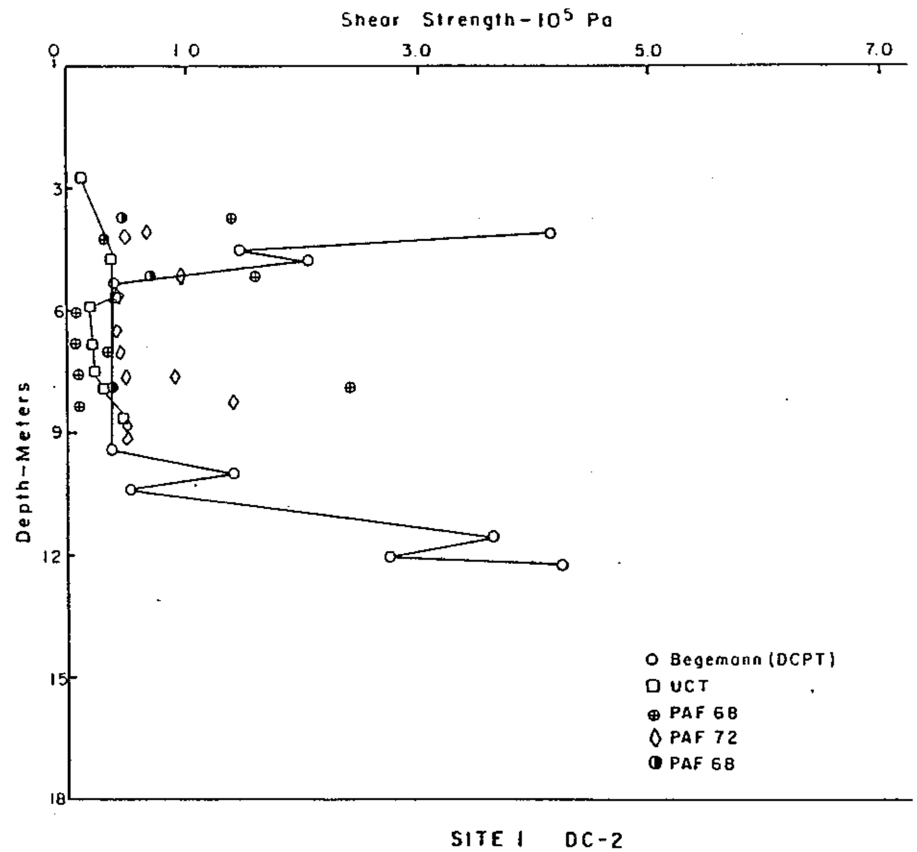


Figure 57 COMPARATIVE STUDY OF SHEAR STRENGTH, (DISCONTINUOUS), SITE 1

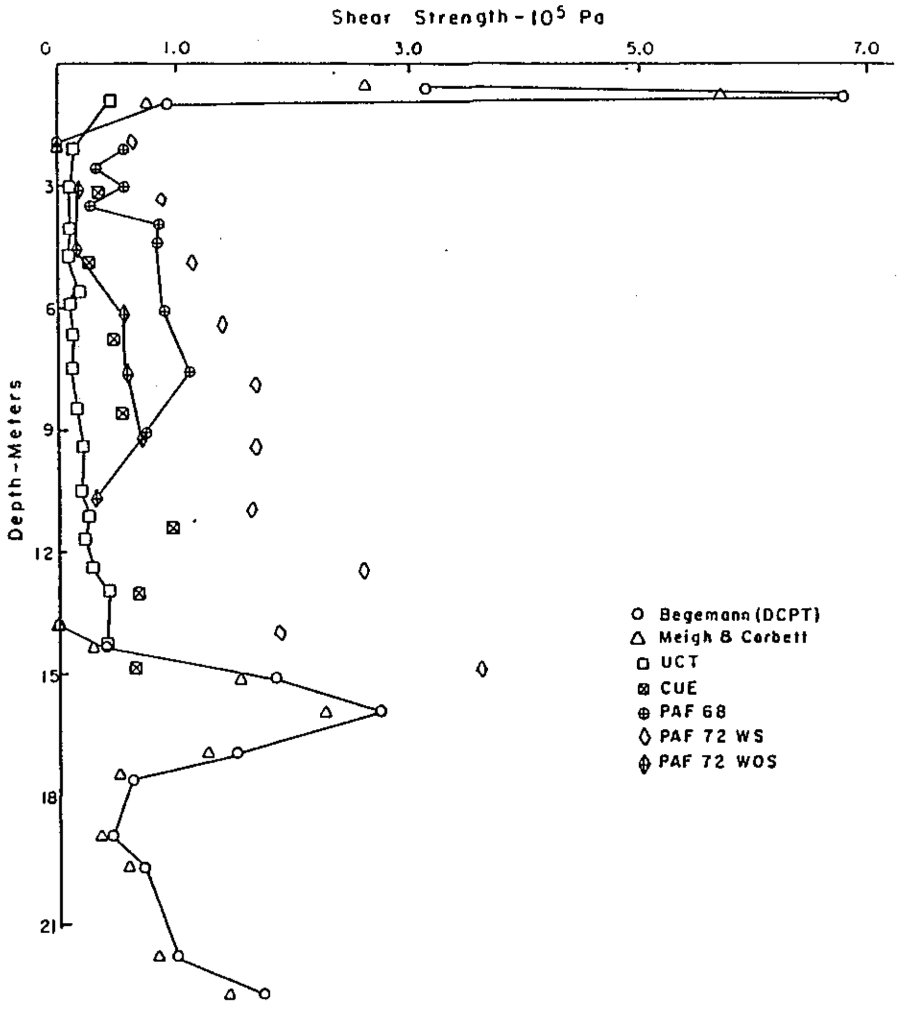


Figure 58 COMPARATIVE STUDY OF SHEAR STRENGTH, (CONTINUOUS), SITE 2

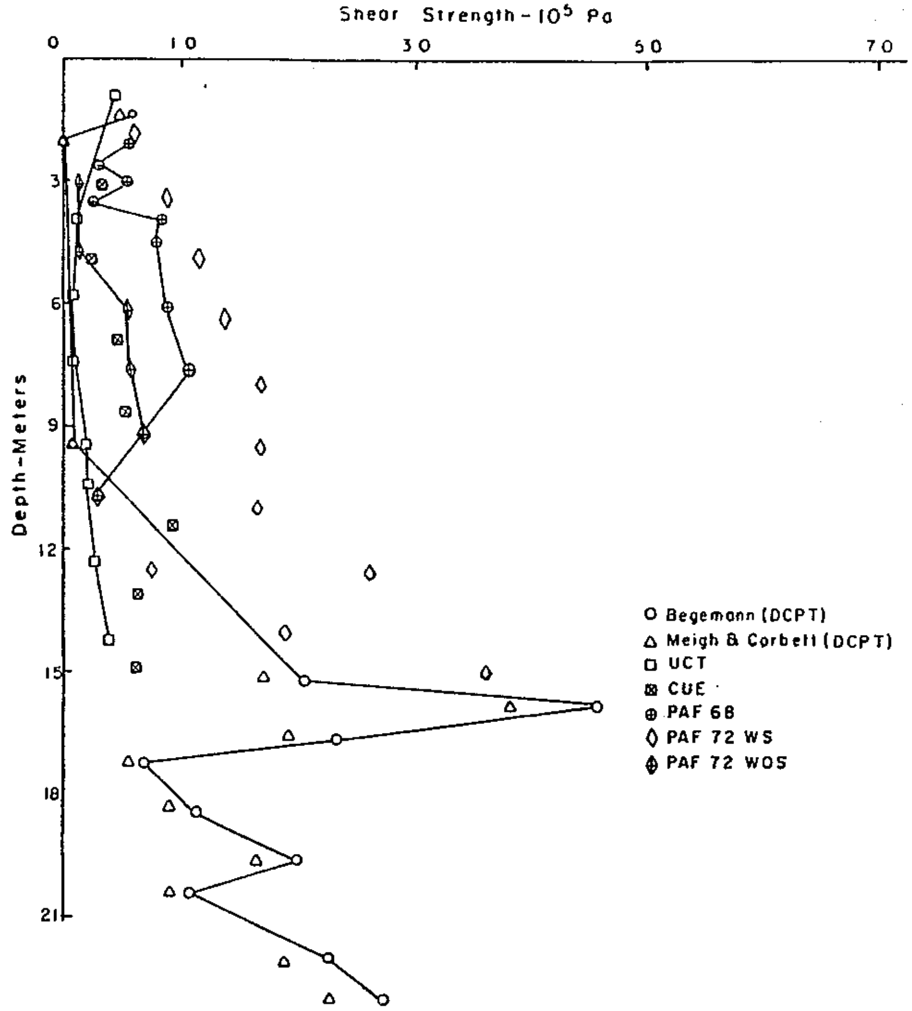


Figure 59 COMPARATIVE STUDY OF SHEAR STRENGTH, (DISCONTINUOUS), SITE 2

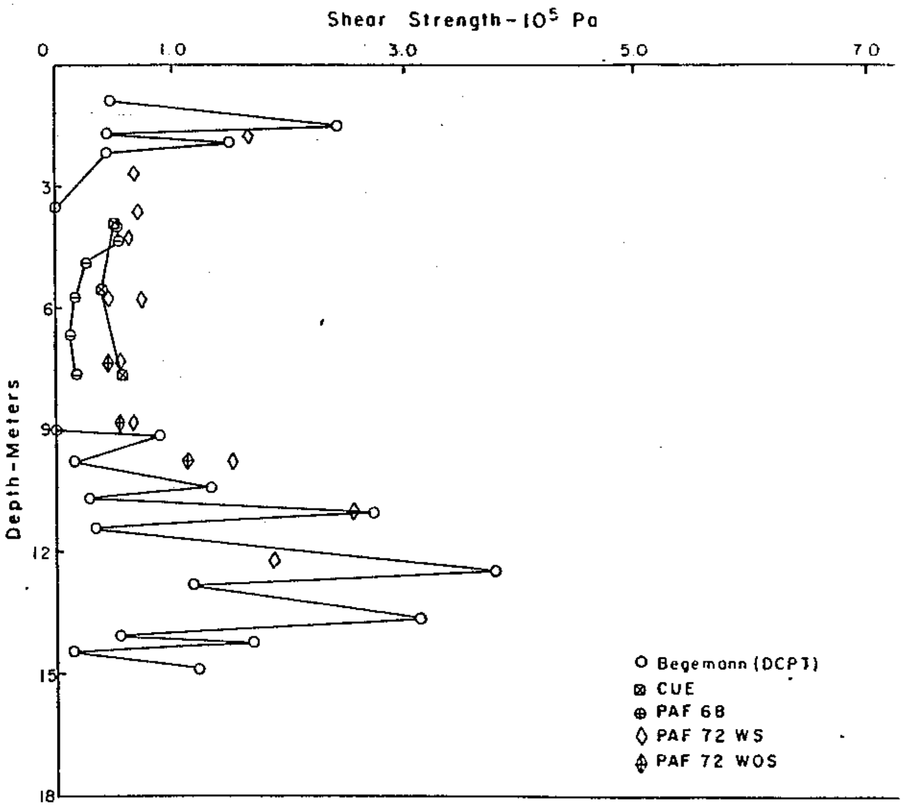


Figure 60 COMPARATIVE STUDY OF SHEAR STRENGTH, (CONTINUOUS), SITE 3

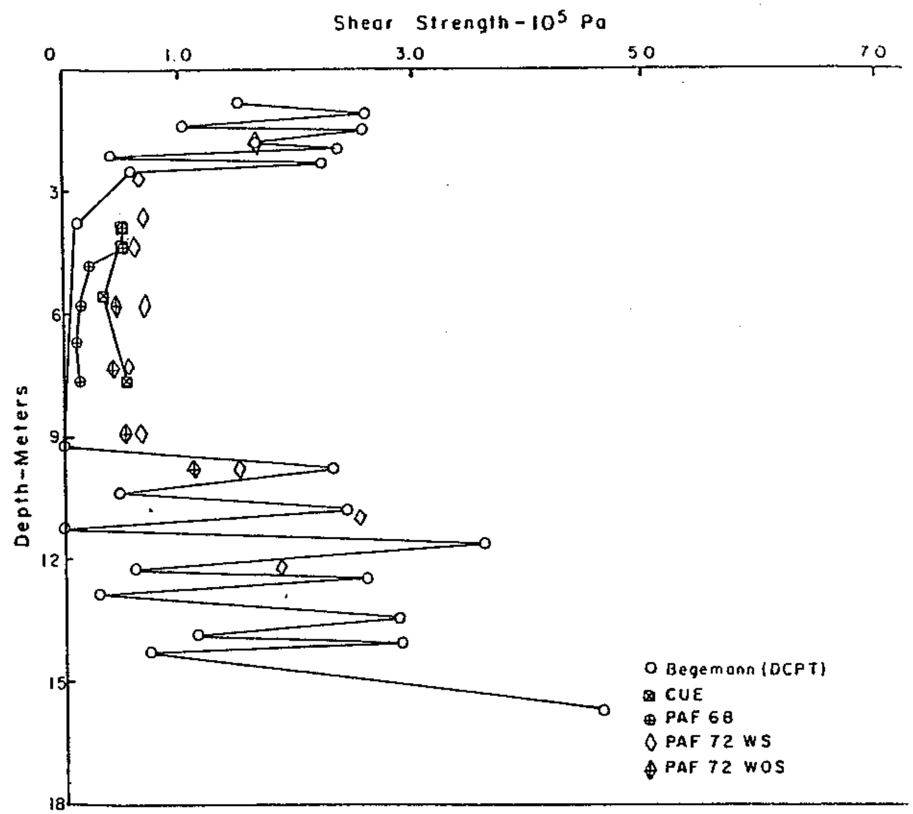


Figure 61 COMPARATIVE STUDY OF SHEAR STRENGTH, (DISCONTINUOUS), SITE 3

the discontinuous method of testing. Depth profiles indicating the variation of shear strength with depth using both methods, with the laboratory as well as the field in situ testing devices, are shown by Figures 56 through 61.

At Site 1, the Begemann factor gave shear strength values higher than any of the other methods. In every case, the Meigh and Corbett factor gave slightly lower values. It may be concluded that these factors require regional revision.

In Reference 2, the following Dutch Cone factors are proposed for four California soils: silty-clay, 4; peaty clay, 5; stiff clay, 12; and silty sand, 17. This suggests that a consistent value cannot be used for these kinds of soils.

Standard Penetrometer

The soils subject to study for this project were primarily soft. The depth profiles of blow counts corrected for overburden pressure for Sites 1 to 3 are presented in Figures 62 through 64. An attempt was made to correlate data from the Standard Penetrometer tests with that obtained with the Dutch Cone Penetrometer. Standard Penetrometer blow count values were modified by taking into account the overburden pressure. These are the values plotted in Figures 62 through 64. To compare Standard Penetrometer test values with those from the Dutch Cone, the unmodified blow count number was divided by the cone resistance from the Dutch Cone and plotted along the Y-axis, with the friction ratio percent plotted along the X-axis. These plots are presented in Figures 65 and 66. From these plots, the N values can be predicted knowing q_c and friction ratio values.

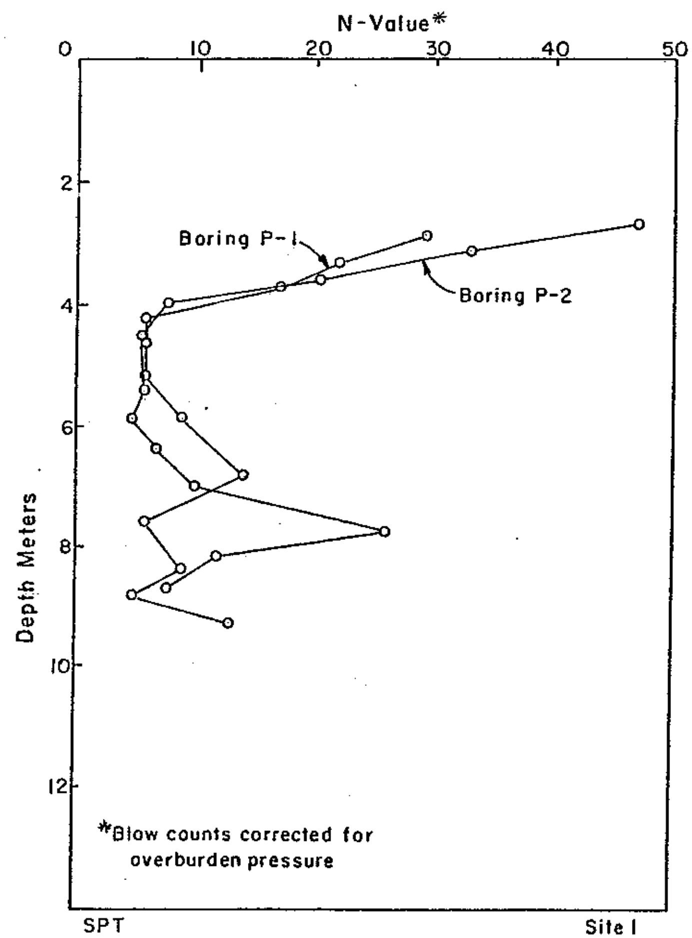


Figure 62 DEPTH PROFILES OF BLOW COUNTS, SITE 1

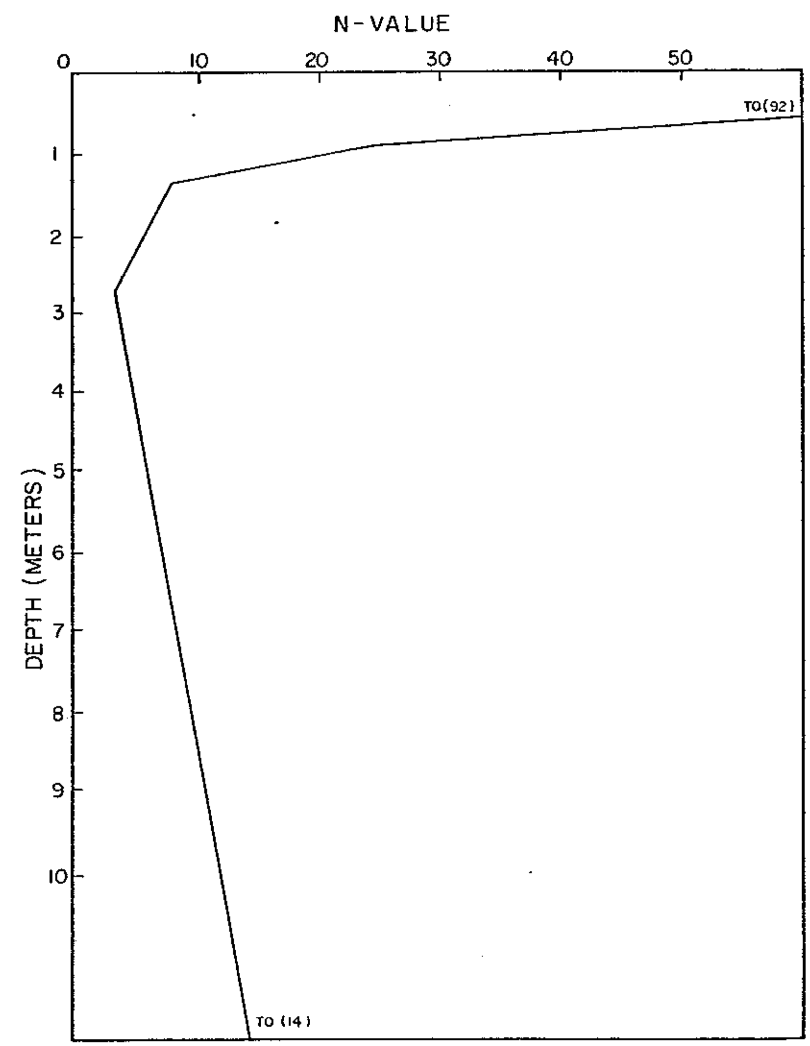


Figure 63 DEPTH PROFILES OF BLOW COUNTS, SITE 2

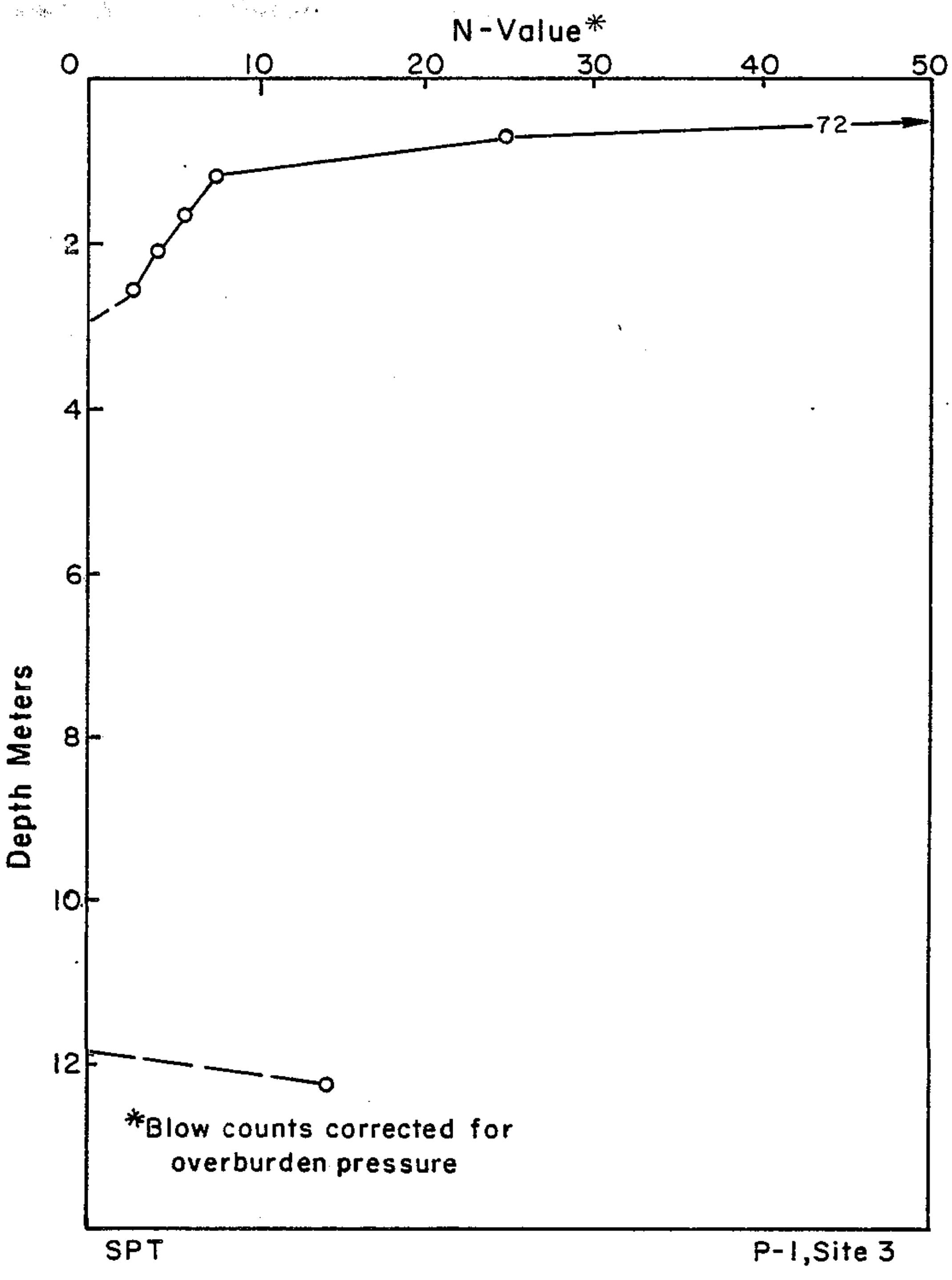


Figure 64 DEPTH PROFILES OF BLOW COUNTS, SITE 3

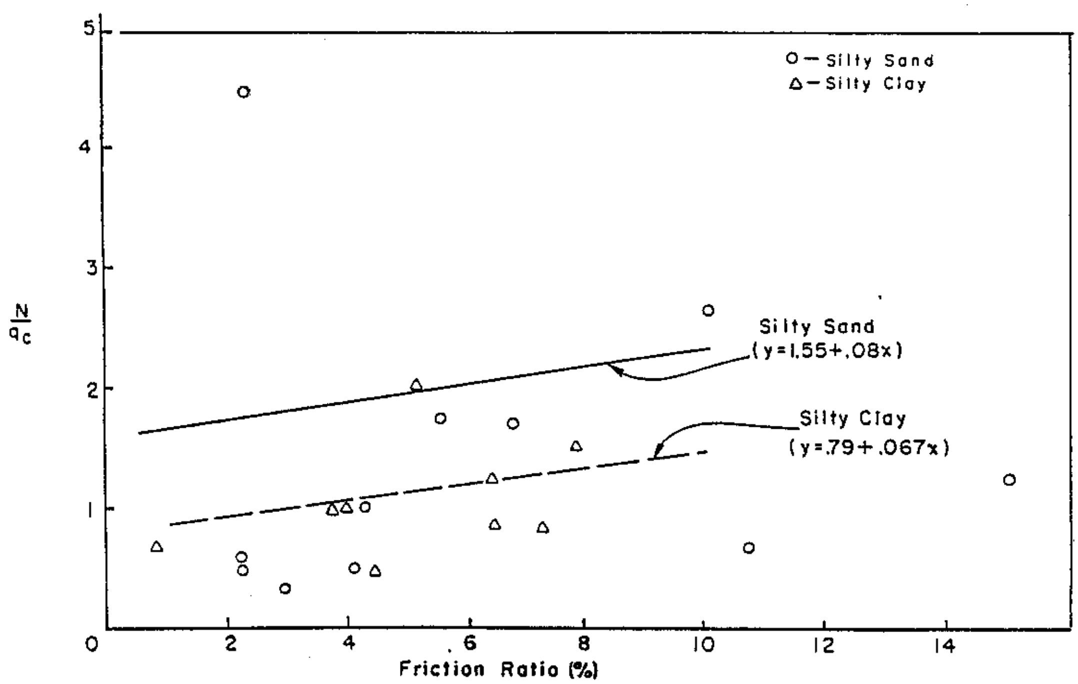


Figure 65 N/q_c VS FRICTION RATIO, SITE 1

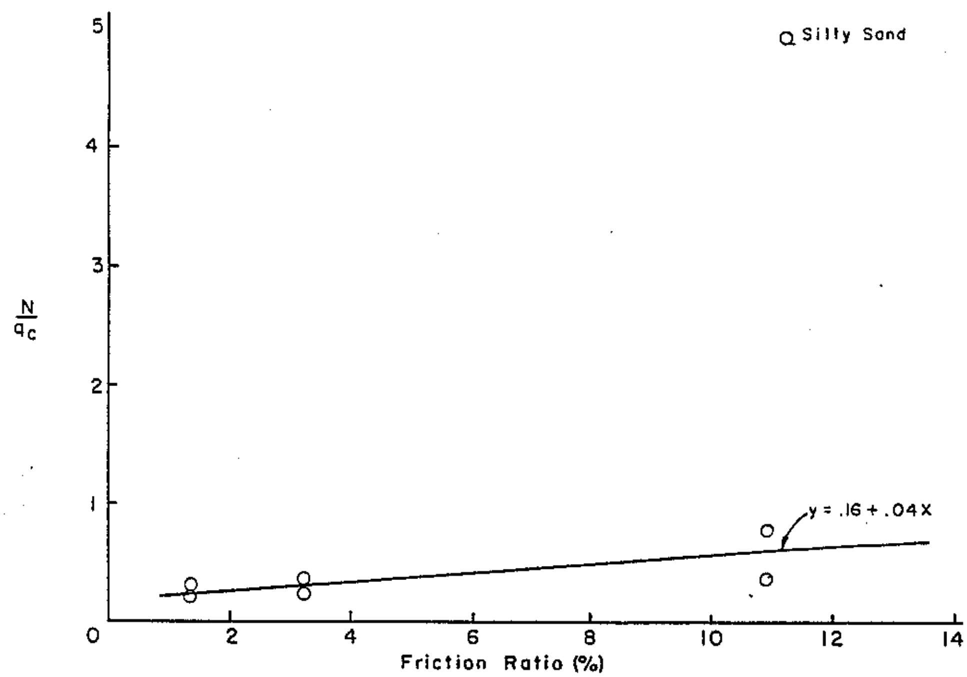


Figure 66 N/q_c VS FRICTION RATIO, SITE 2

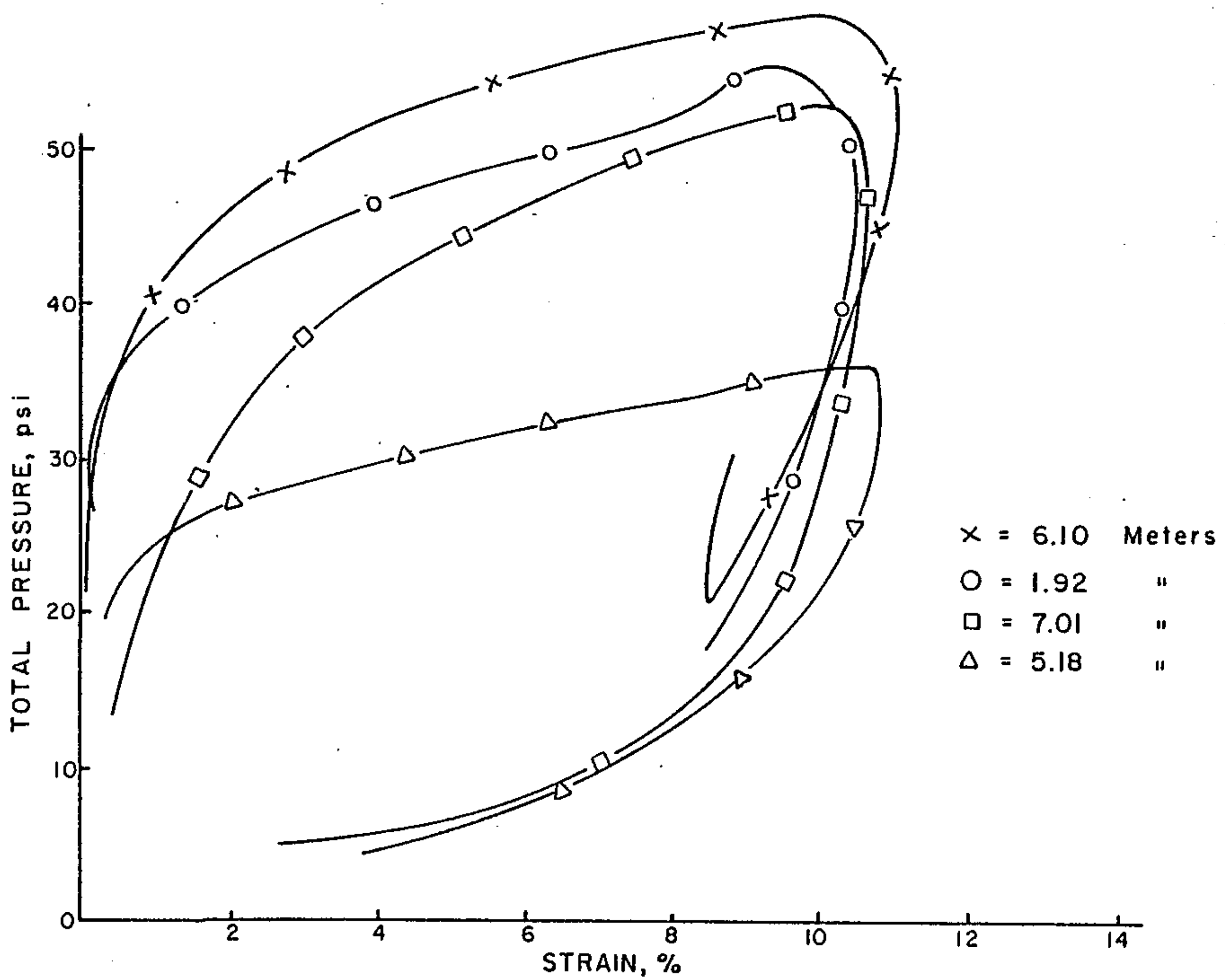


Figure 67 CAMBRIDGE PRESSUREMETER CURVES, SITE 1

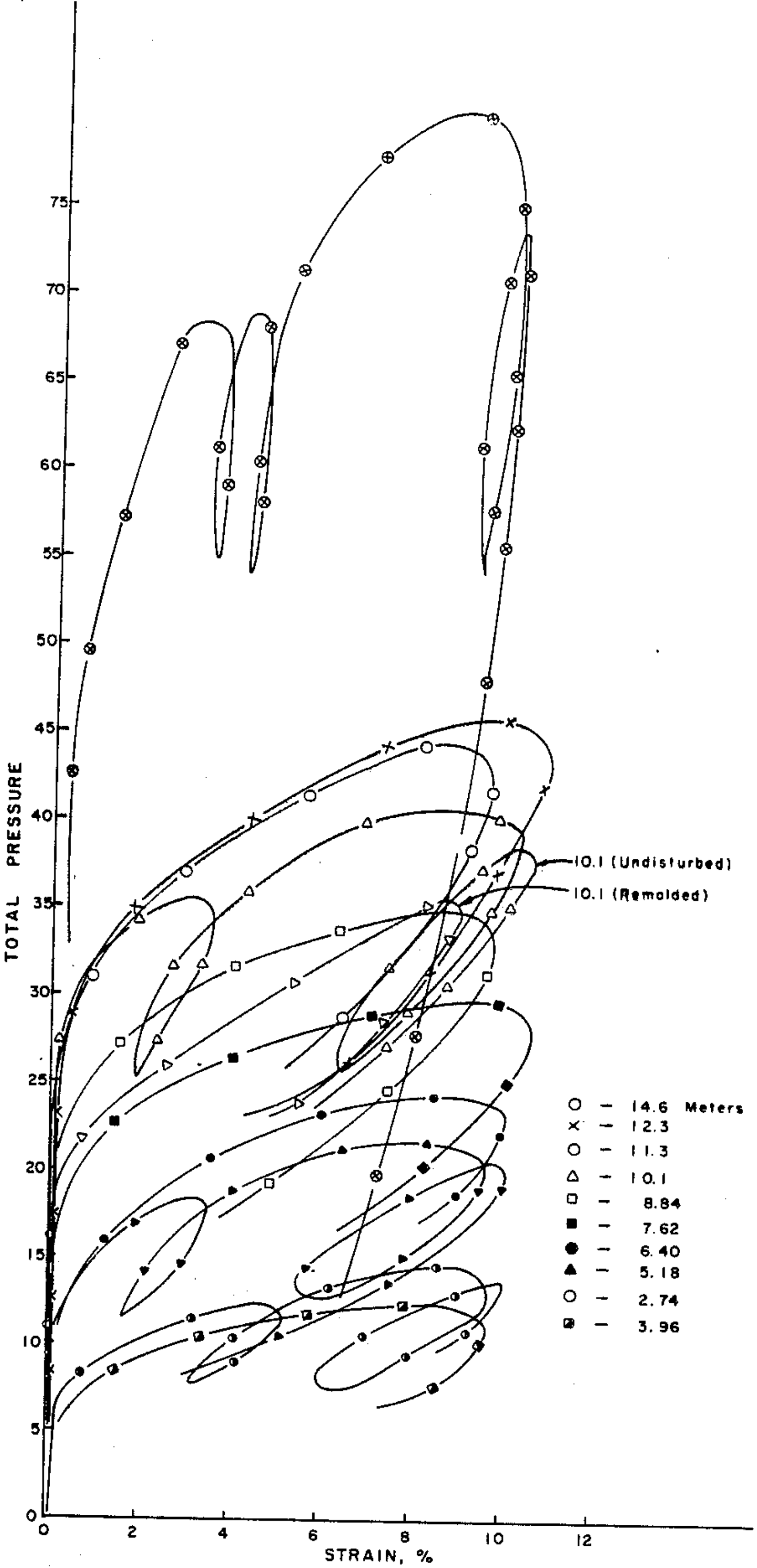


Figure 68 CAMBRIDGE PRESSUREMETER CURVES, SITE 2

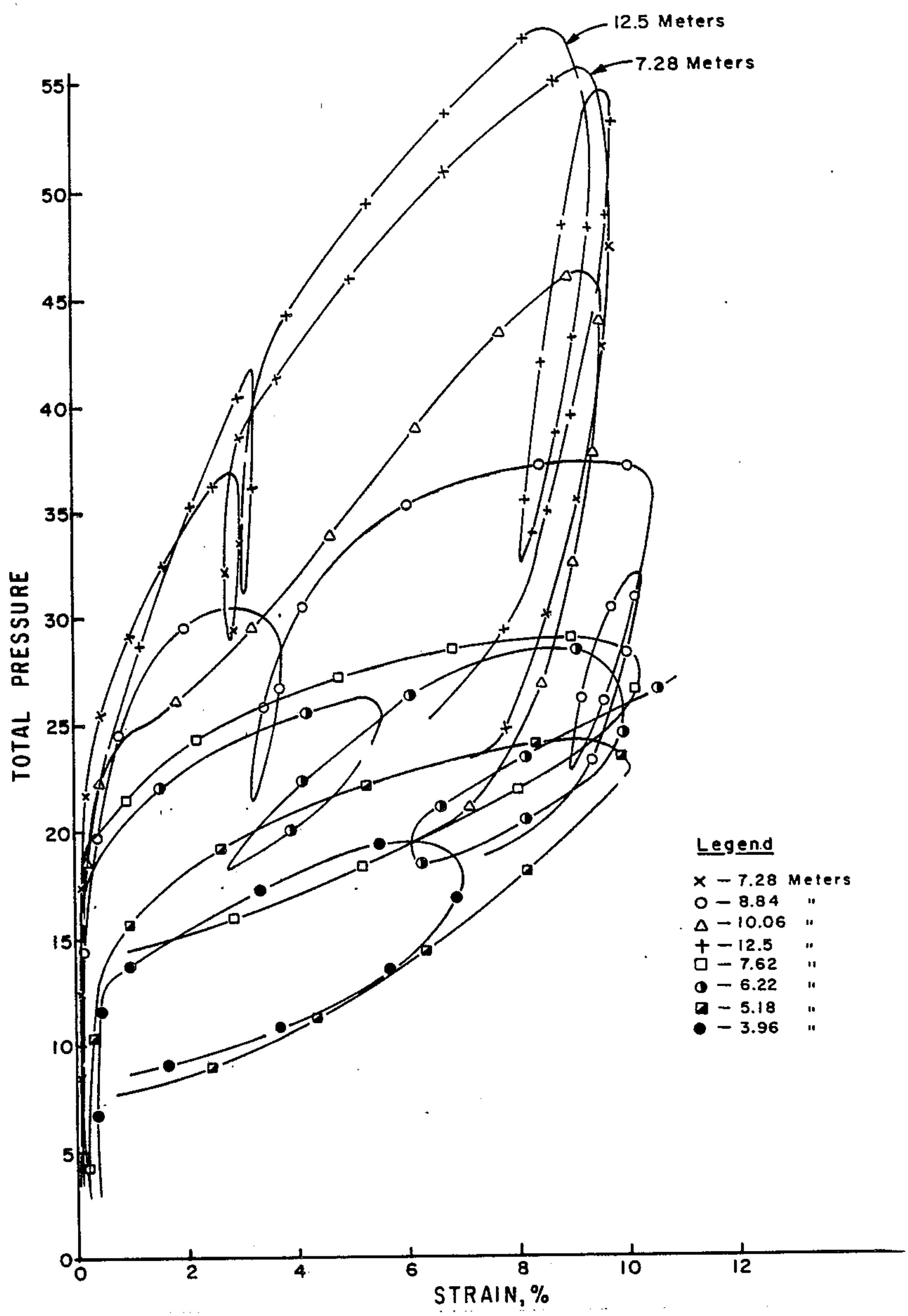


Figure 69 CAMBRIDGE PRESSUREMETER CURVES, SITE 3

Cambridge Self Boring Pressuremeter

The test results from the Cambridge self boring pressuremeter were used to derive various parameters for use in geotechnical design. The corrected Cambridge pressuremeter curves for Sites 1 to 3 are presented in Figures 67 through 69.

The shear strength values derived from the Cambridge self boring pressuremeter test data were compared with shear strength parameters obtained from either laboratory tests or other field in situ tests. Figure 70 depicts a number of depth profiles that developed for Site 1. In Figure 70(A) the peak undrained shear strength derived from the Cambridge pressuremeter is compared with shear strength derived from Dutch Cone test data using the Begemann factor of 13.6 and the shear strength derived using the vane. As can be seen, the Cambridge Pressuremeter shear strength is very close to that obtained using the vane. The shear strength using the Dutch Cone data is generally much lower than that measured using the Cambridge probe.

The strength values using the Torvane and the hand penetrometer have been plotted in Figure 70(B). Here, the hand penetrometer strength values were far higher than those from the Cambridge probe. The Torvane values in general were much less than the Cambridge shear strength values.

In Figure 70(C) the shear strength derived using the Cambridge probe is compared with the shear strength derived using the French pressuremeters. Again, in general it can be seen that shear strengths from the French pressuremeters were generally higher than those derived from the Cambridge probe.

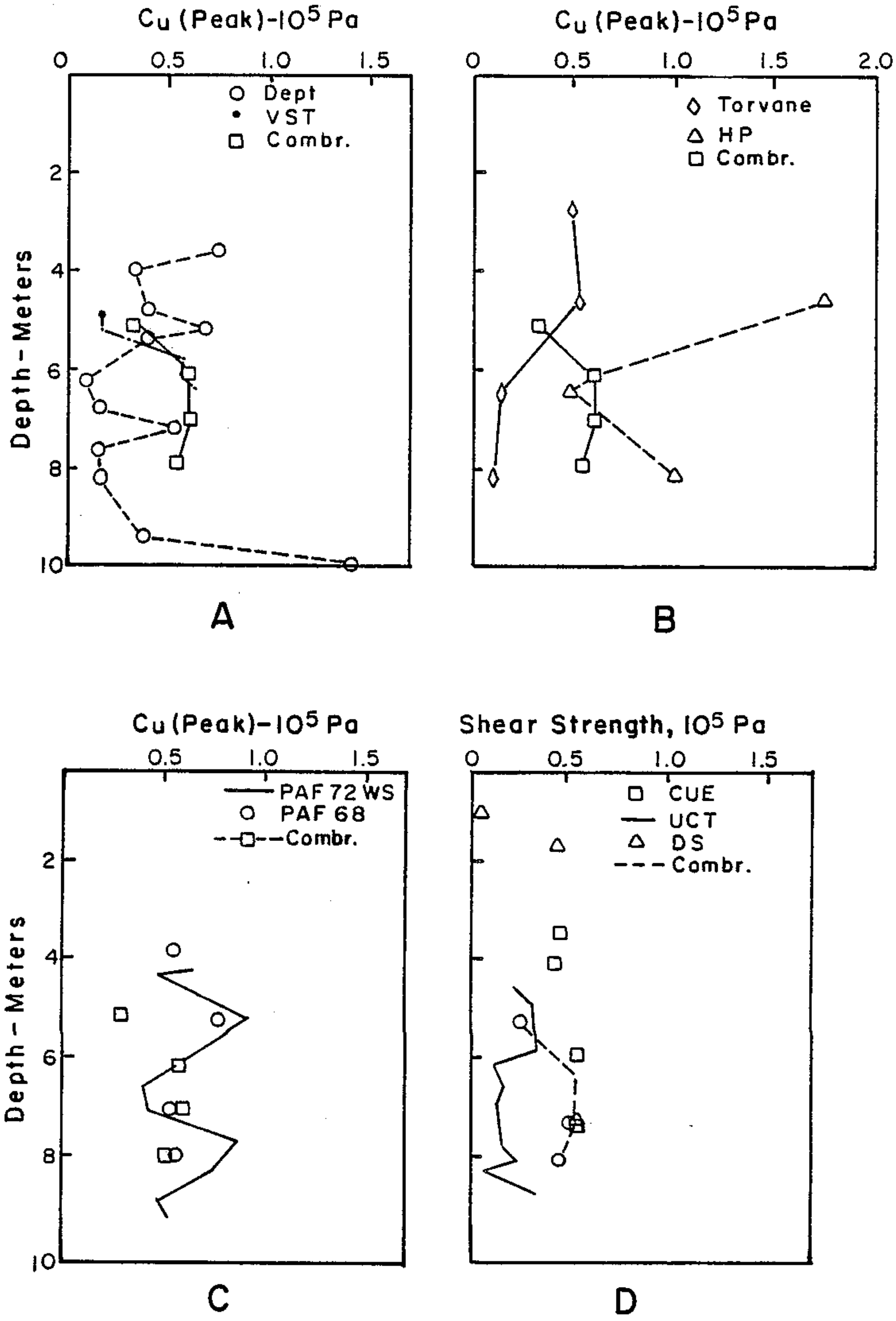


Figure 70 COMPARISON OF SHEAR STRENGTH, SITE 1

In Figure 70(D), the Cambridge probe values are plotted against values from the laboratory shear strength tests. In general, the Cambridge probe values are between the unconfined compressive strength tests and the direct shear tests.

Sensitivity values calculated from vane shear tests, unconfined compressive strength tests, and pressuremeter tests are presented in Figure 71. The coefficient of earth pressure at rest (K_o) values were determined using Wroth's formulae, Jaky's formula, Alpan's formula, and from pressuremeter test data. These values are plotted in Figure 72. The various moduli [subtangent moduli (E_s) and failure moduli (E_f)] from pressuremeter data are depicted in Figure 73. Various limit pressure values are summarized in Figure 74. The P_L (Theoretical Limit pressure) values by the practical method are presented in Figure 74(A) and those by the Marsland and Randolph (M&R) approach in Figure 74(B).

As can be seen from Figure 75(A) there were very few shear strength values obtained from Dutch Cone data between depths of about 2 and 9 meters at Site 2. In general, the shear strength values from the Dutch Cone were much lower than those from the vane or from the Cambridge probe. Compared to the vane and the Dutch Cone, the Cambridge probe gave the higher peak undrained shear strength.

Figure 75(B) depicts comparison between the shear strength derived from the Cambridge probe with that from the Torvane and the hand penetrometer. Here again, the values from the Cambridge probe were very close to those from the hand penetrometers and much higher than those from Torvane.

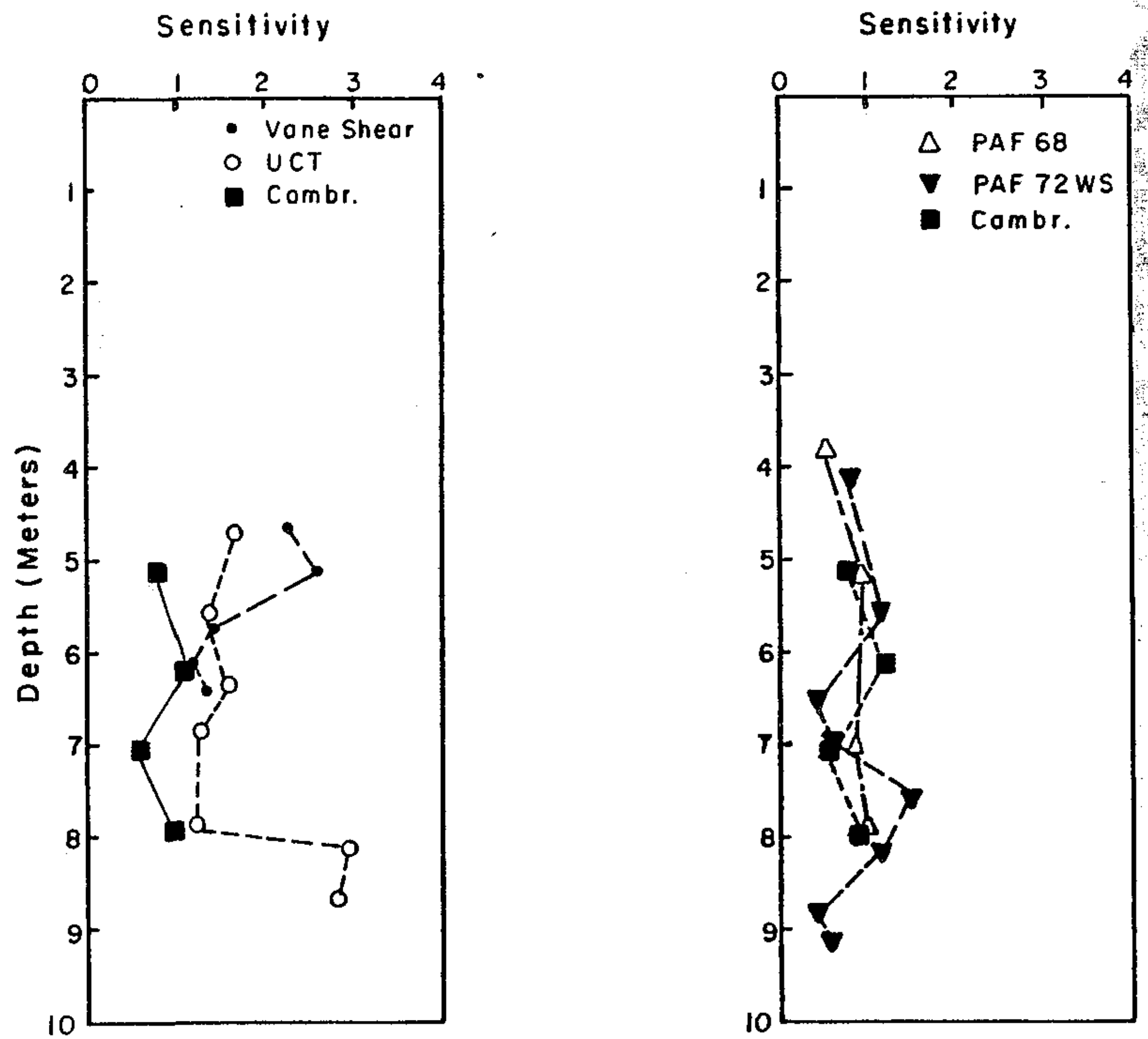


Figure 71 SENSITIVITY STUDY, SITE 1

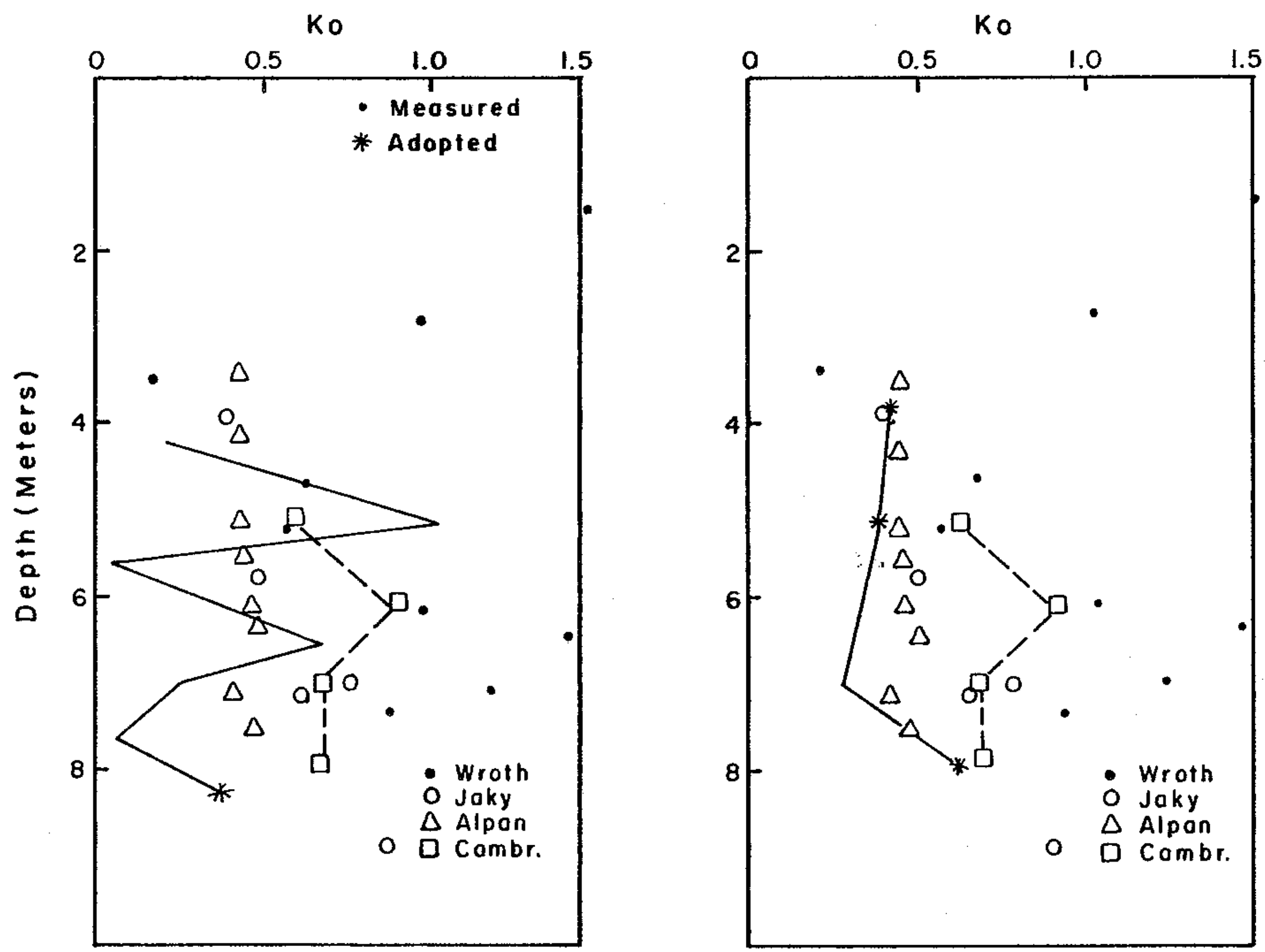


Figure 72 COMPARISON OF K_o, SITE 1

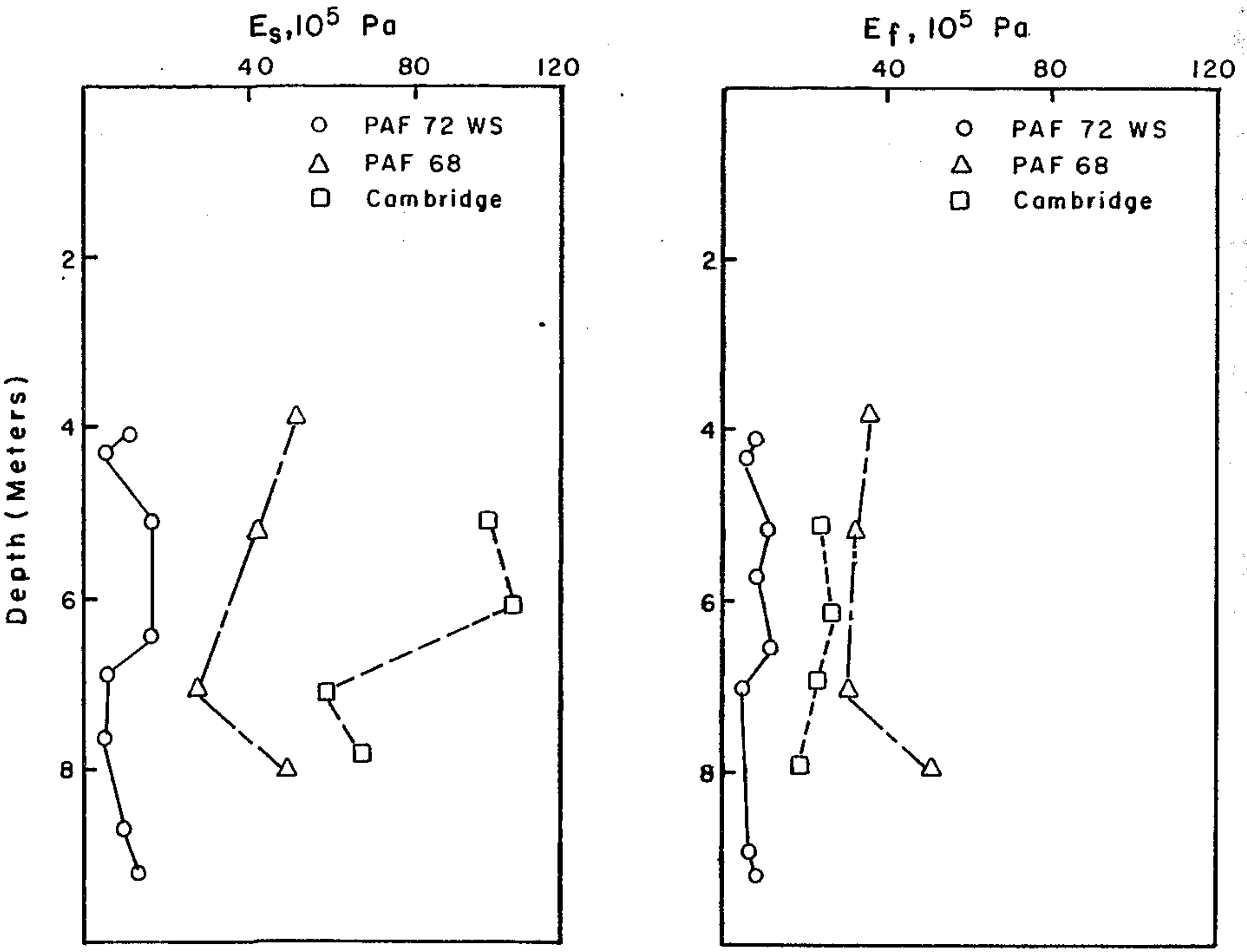


Figure 73 COMPARISON OF VARIOUS MODULI, SITE 1

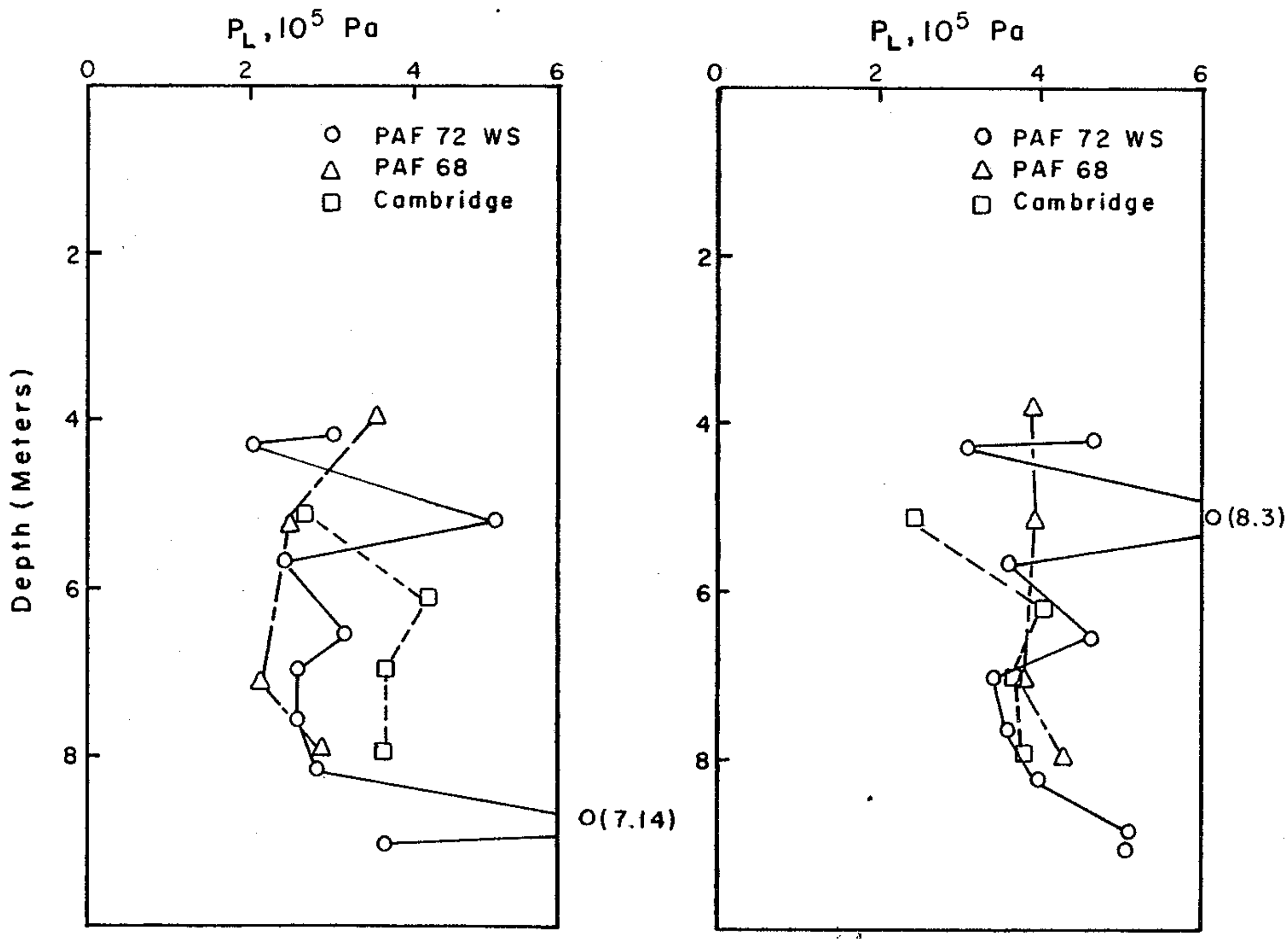


Figure 74 COMPARISON OF LIMIT PRESSURES, SITE 1

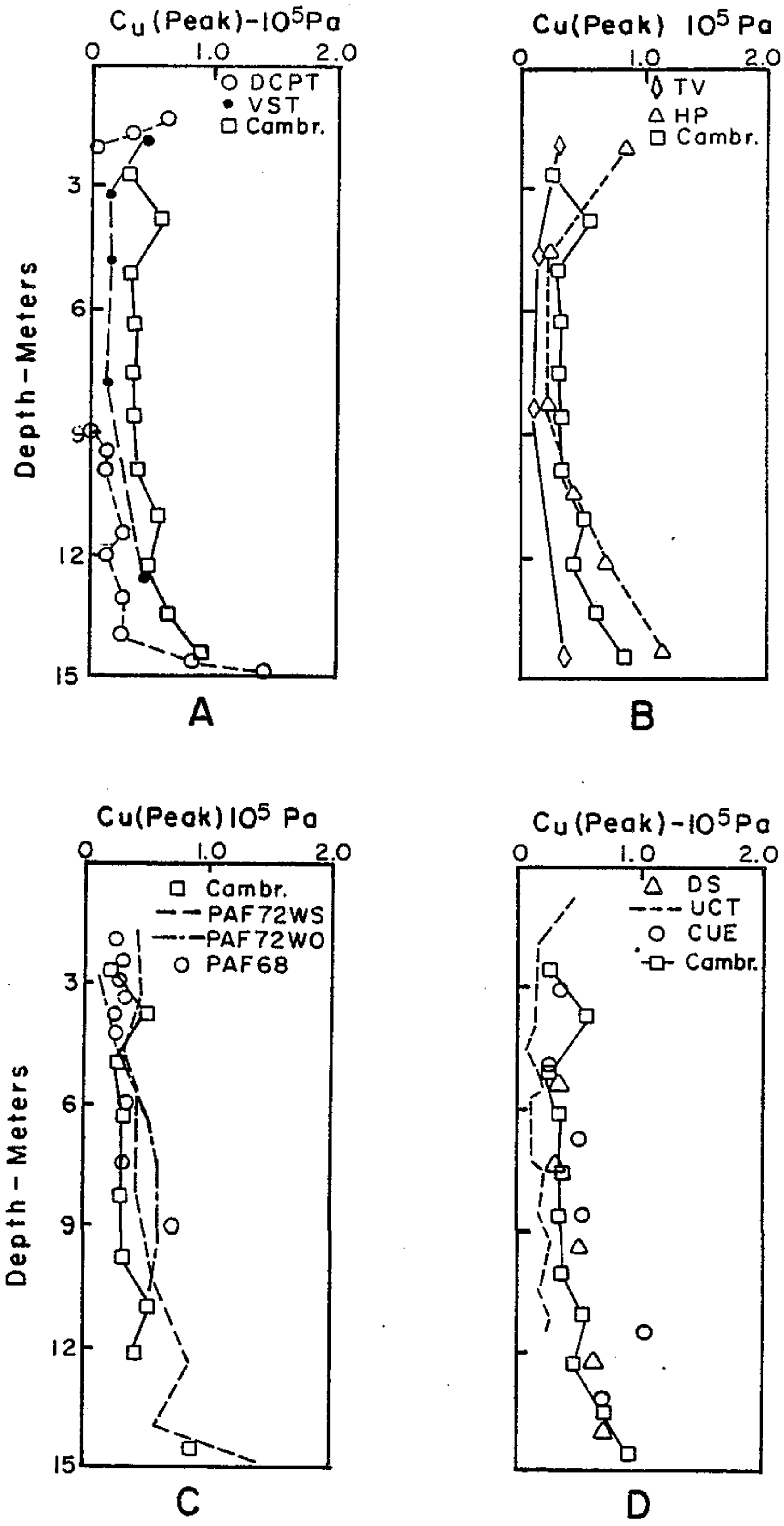


Figure 75 COMPARISON OF SHEAR STRENGTH, SITE 2

In Figure 75(C), Cambridge probe shear strength values are compared with those from the French self boring pressuremeters. Again, it can be observed that the Cambridge probe strength values are fairly close to those from the French self boring pressuremeters.

In Figure 75(D), Cambridge probe shear strength values are compared to laboratory values. Here, also, the values from the Cambridge probe are very close to those obtained in the laboratory.

Sensitivity values are presented in Figure 76. Values from the Cambridge probe are lower than those from vane and unconfined tests. When compared to the French probes also, Cambridge probe values were lower. The K_o plots are presented in Figure 77. The K_o values predicted using the Cambridge probe data are in the same general range as those from French pressuremeters.

A perusal of the various moduli [initial tangent moduli (E_o), subtangent moduli (E_s) and failure moduli (E_f)] presented in Figure 78 would indicate the Cambridge and French probes yield values which are similar. Even the various limit pressures (both practical and theoretical) presented in Figure 79 indicate comparable results for the French and Cambridge probes.

The shear strength depth profiles for Site 3 are presented in Figure 80. Shear strength values from the Cambridge probe are compared with the Dutch Cone and the vane, in Figure 80(A). Again, the Dutch Cone values were the lowest and the Cambridge probe values the highest, with the vane shear strength lying between. In Figure 80(B) Torvane values were higher than both hand penetrometers and Cambridge values.

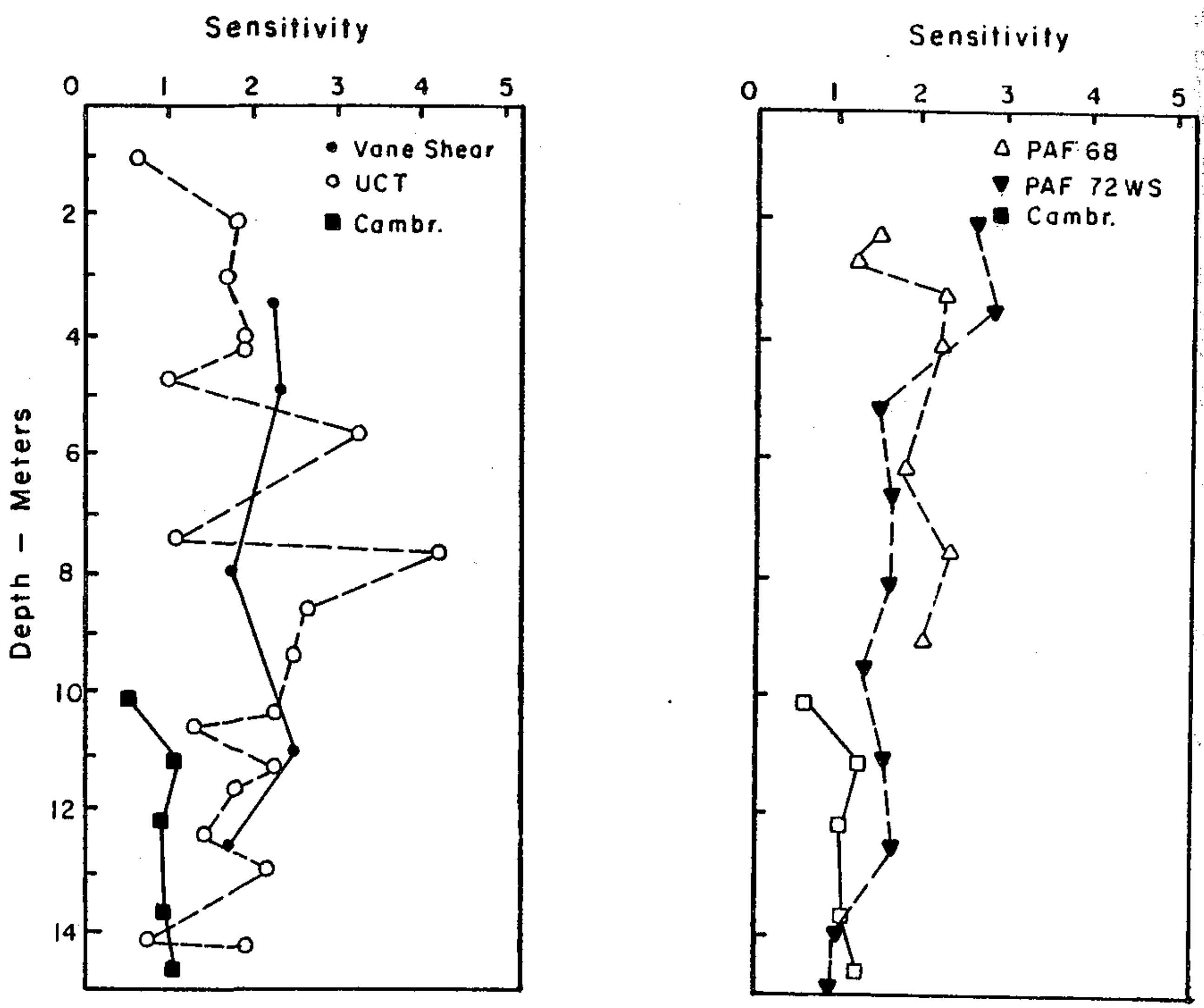


Figure 76 SENSITIVITY STUDY, SITE 2

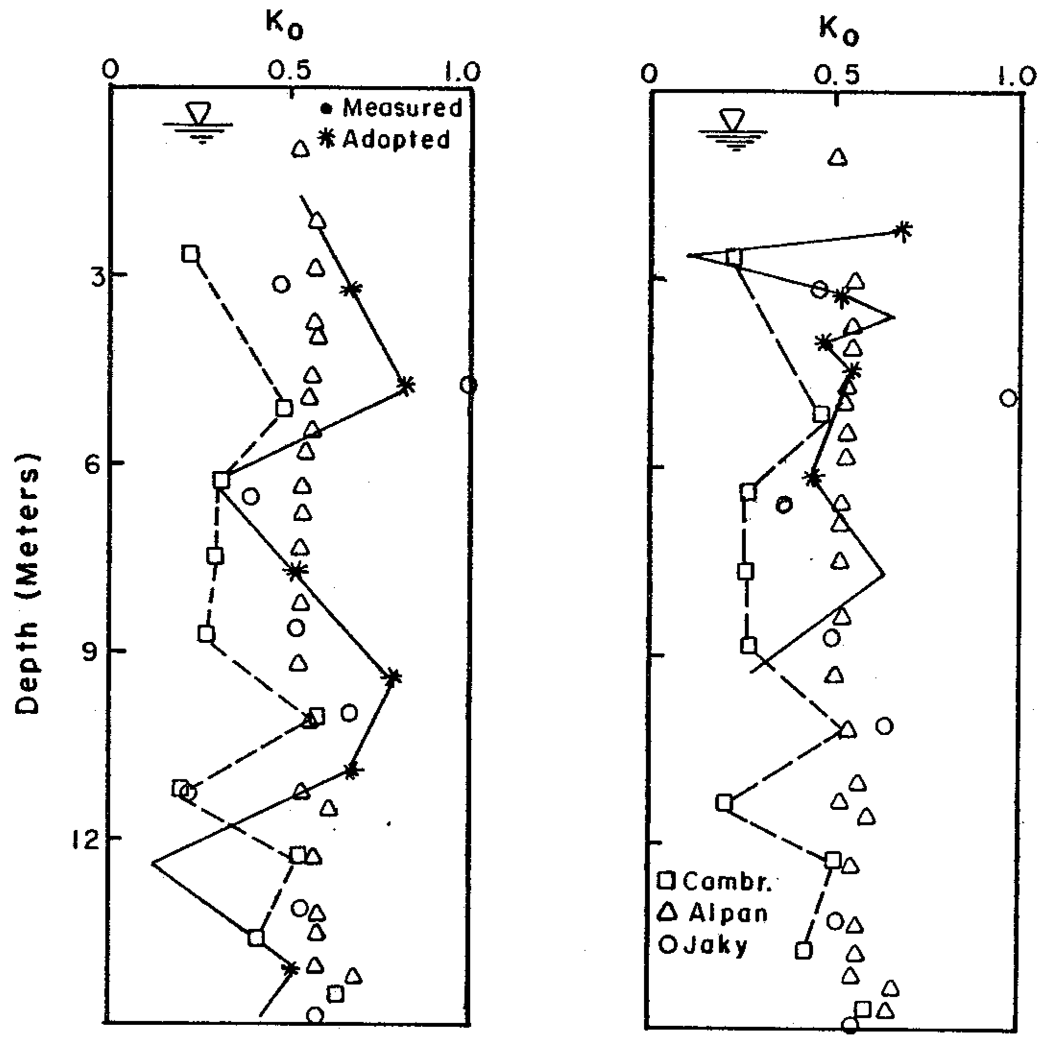


Figure 77 COMPARISON OF K_o, SITE 2

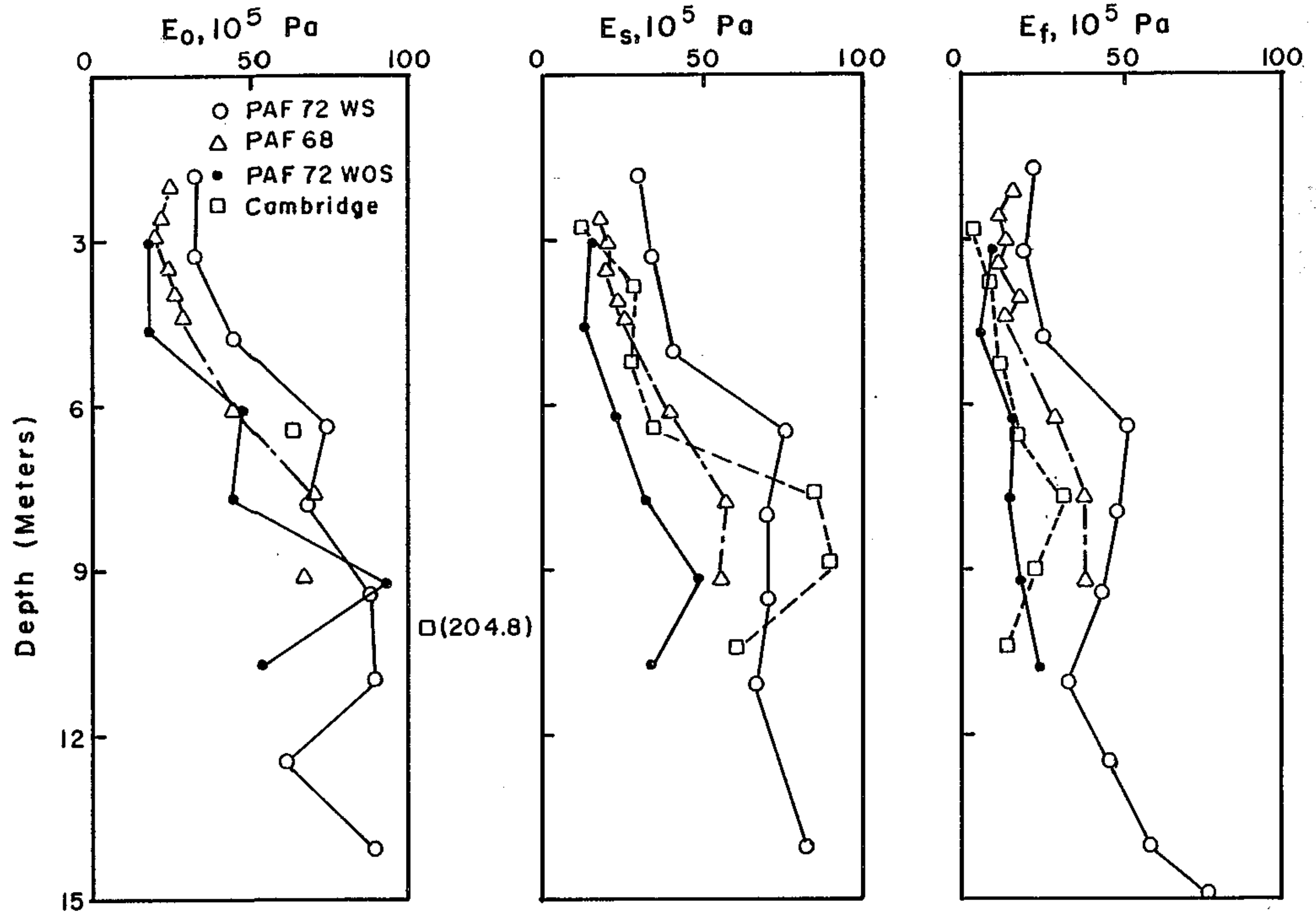


Figure 78 COMPARISON OF VARIOUS MODULI, SITE 2

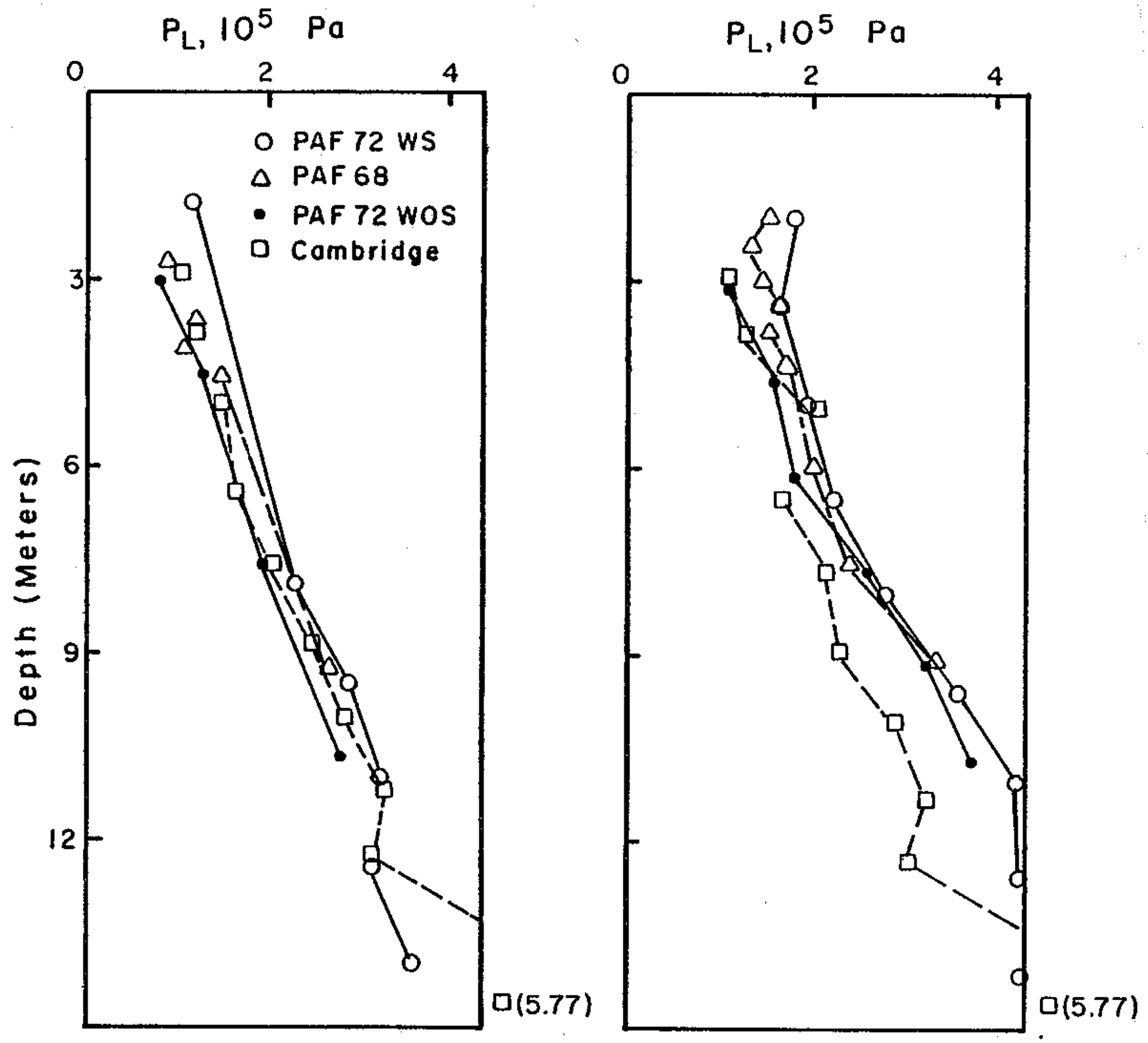


Figure 79 COMPARISON OF LIMIT PRESSURES, SITE 2

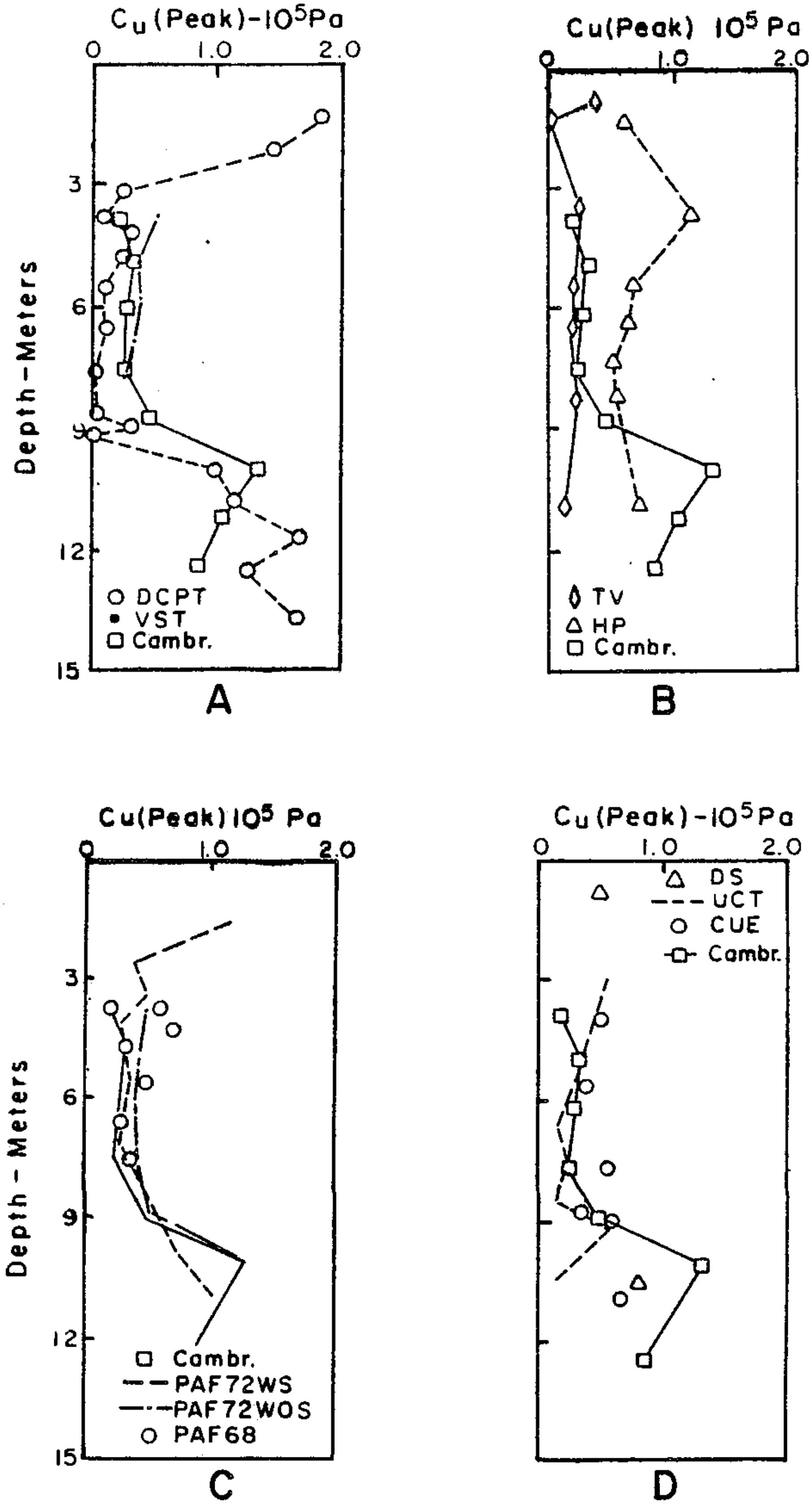


Figure 80 COMPARISON OF SHEAR STRENGTH, SITE 3

In Figure 80(C) the values from the Cambridge probe were in the same range as those from the French probes. In Figure 80(D) the laboratory shear strength values were very similar to the Cambridge values except at depths greater than nine meters, where stiffer clays were encountered.

The sensitivity values (Figure 81) from the Cambridge probe are lower than those from the vane and unconfined laboratory tests; whereas, those from the French probes and Cambridge probes are similar. The K_o values (Figure 82) by the French probes and the Cambridge probe are very similar. The moduli (Figure 83) as well as limit pressures (Figure 84) by both the French and Cambridge probes are in the same general range.

Correlations

An attempt was made to correlate the following: (i), K_o and PI (Plasticity Index); (ii), O.C.R. (over consolidation ratio) and PI; and (iii), K_o and O.C.R.

The correlations were conducted using both linear and non-linear regression analysis for all three sites. These plots are presented in Figures 85 through 93. Using Figures 85, 86, and 87 from known values of P.I., values of K_o can be estimated. Using Figures 88, 89, and 90 from known values of P.I., values of O.C.R. can be estimated. From Figures 91, 92, and 93 from known values of O.C.R., values of K_o can be estimated.

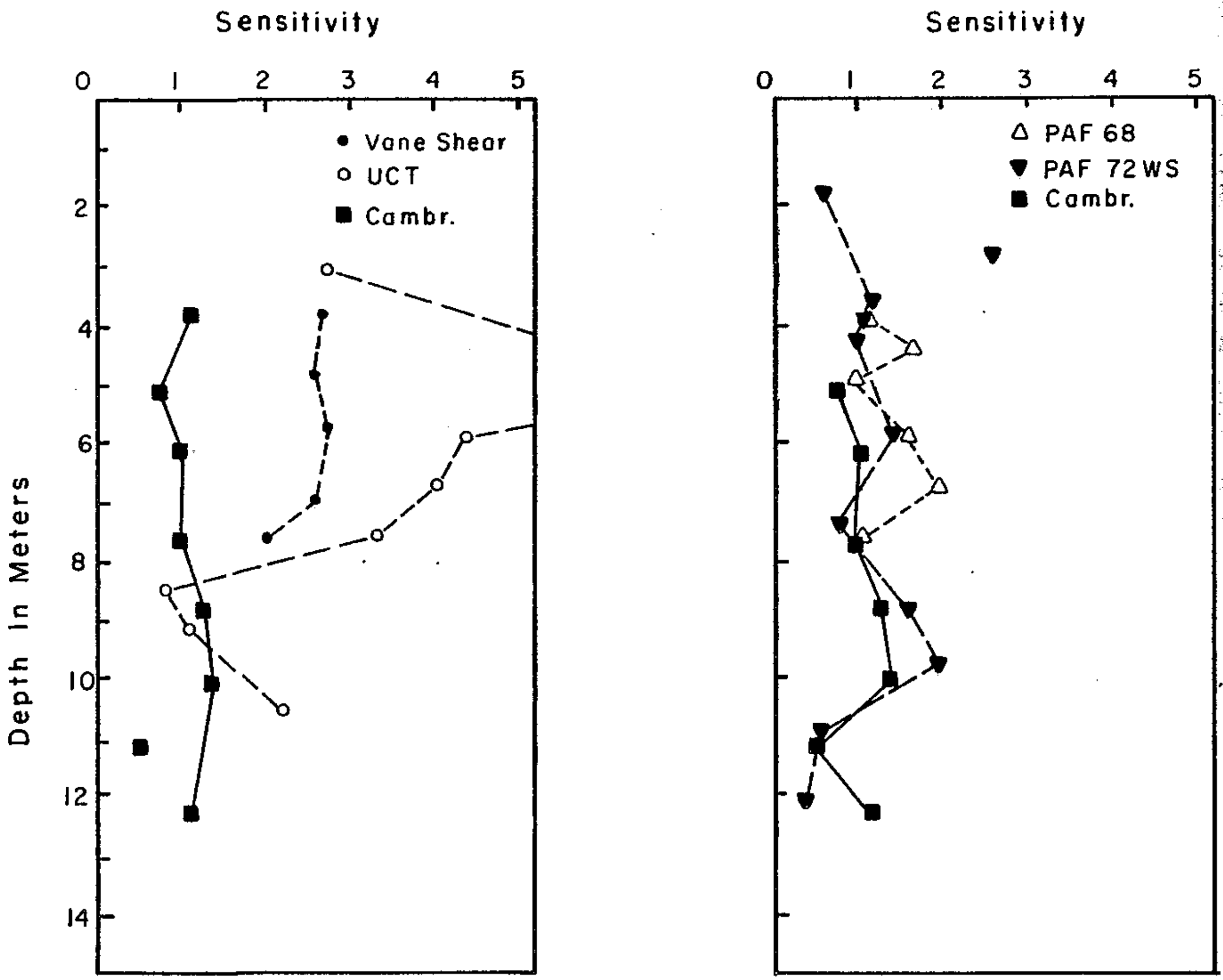


Figure 81 SENSITIVITY STUDY, SITE 3

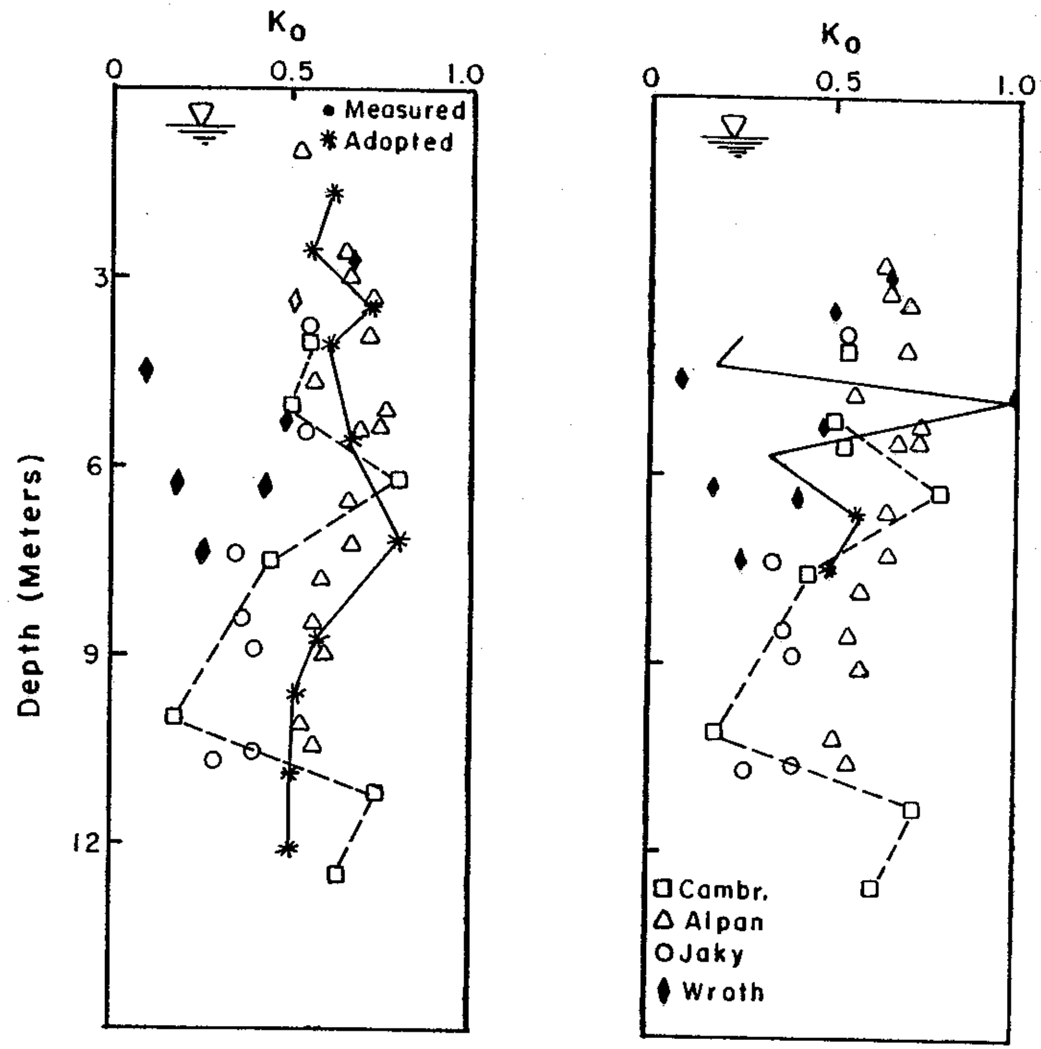


Figure 82 COMPARISON OF K_o, SITE 3

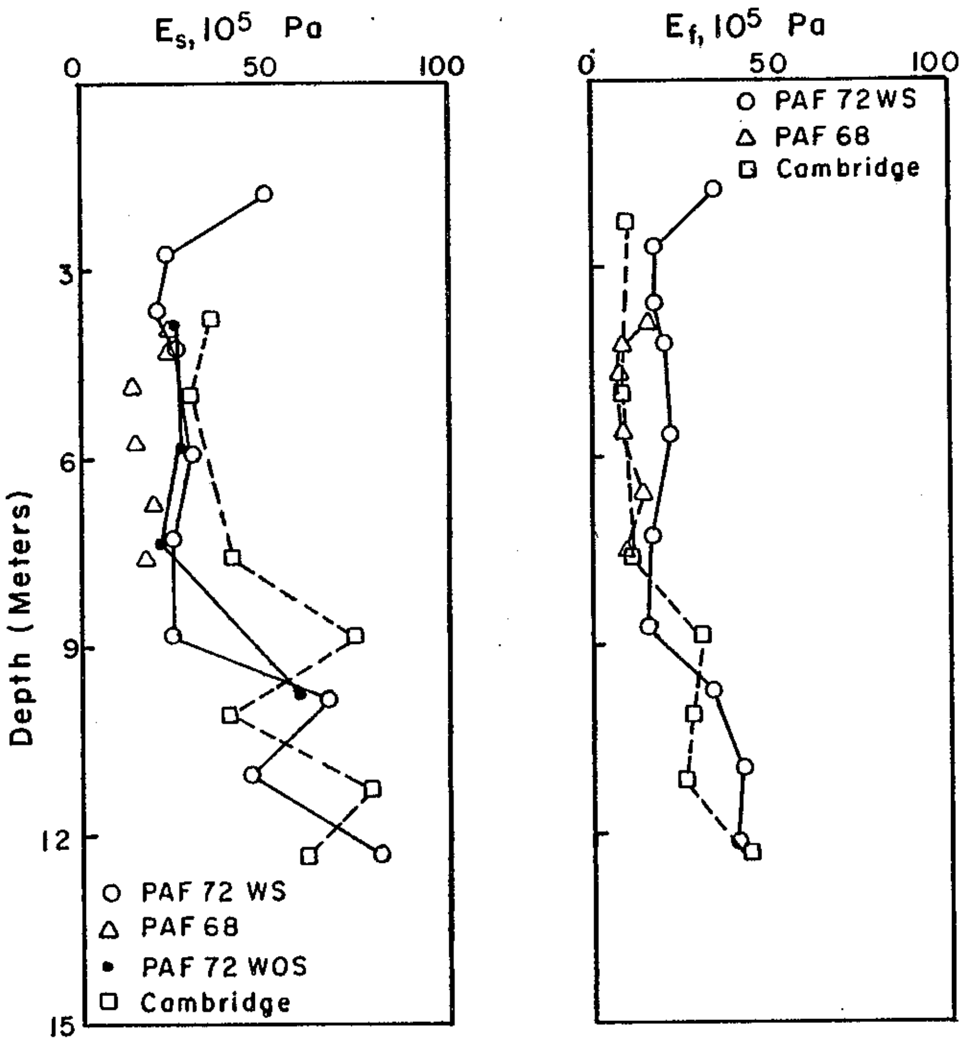


Figure 83 COMPARISON OF VARIOUS MODULI, SITE 3

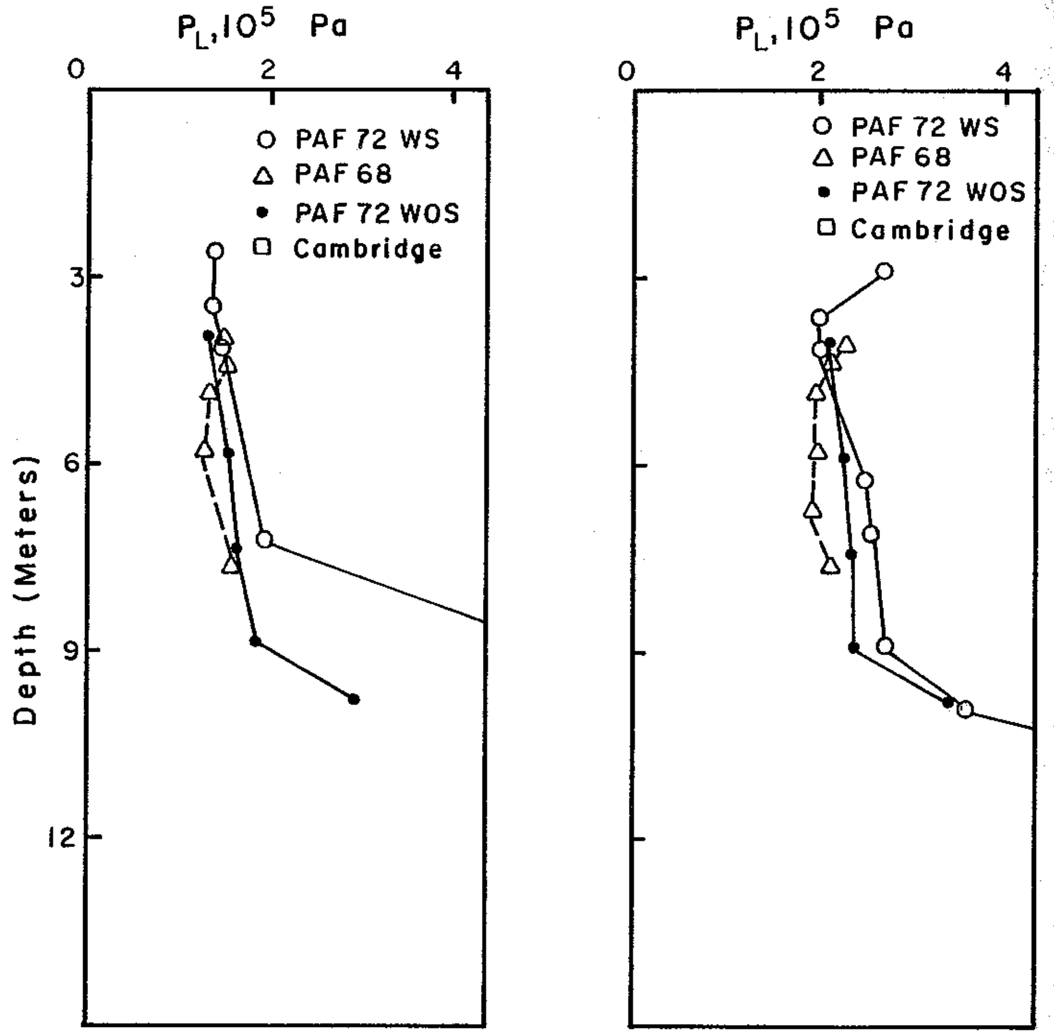


Figure 84 COMPARISON OF LIMIT PRESSURES, SITE 3

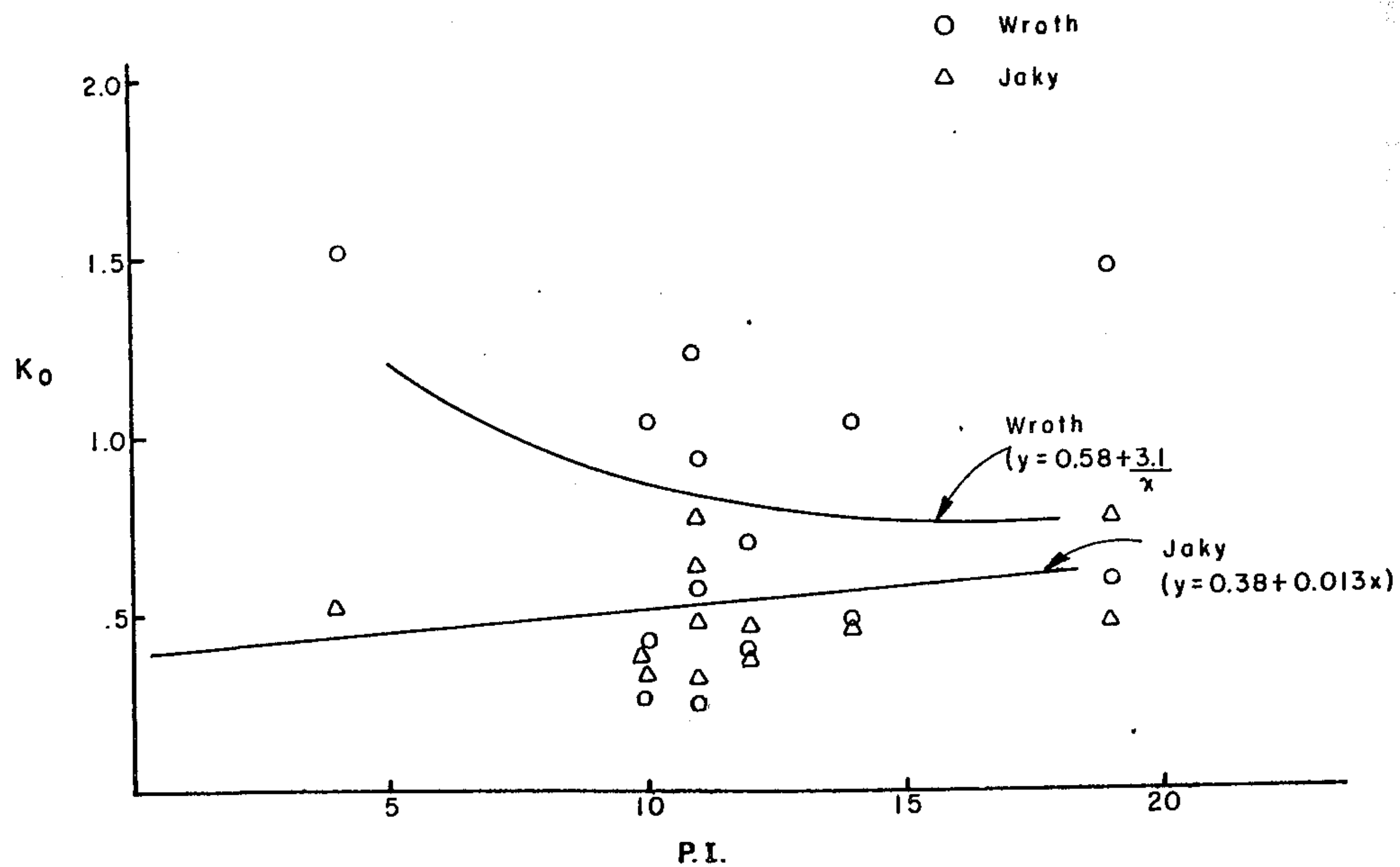


Figure 85 K_0 VS PI, SITE 1

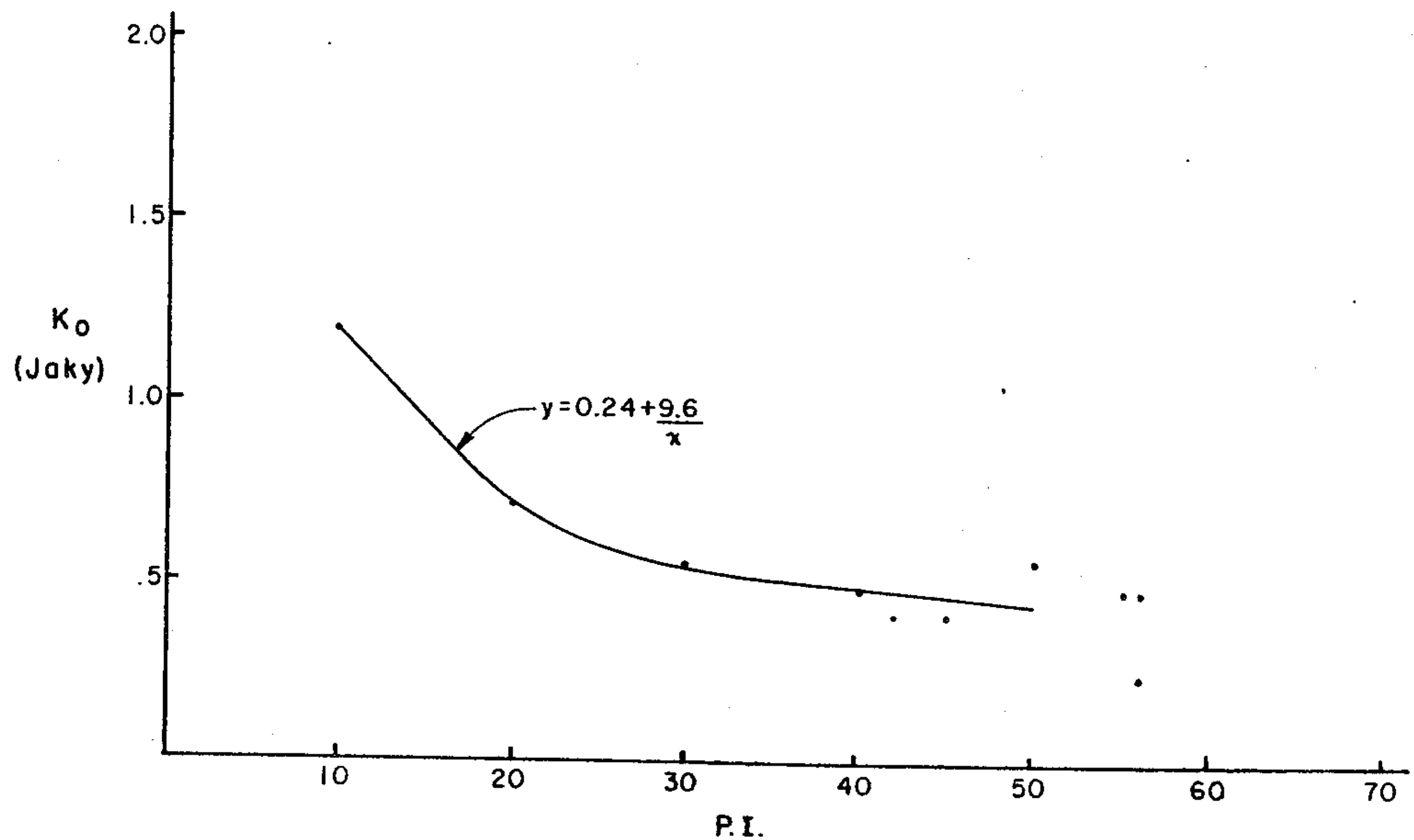


Figure 86 K_0 VS PI, SITE 2

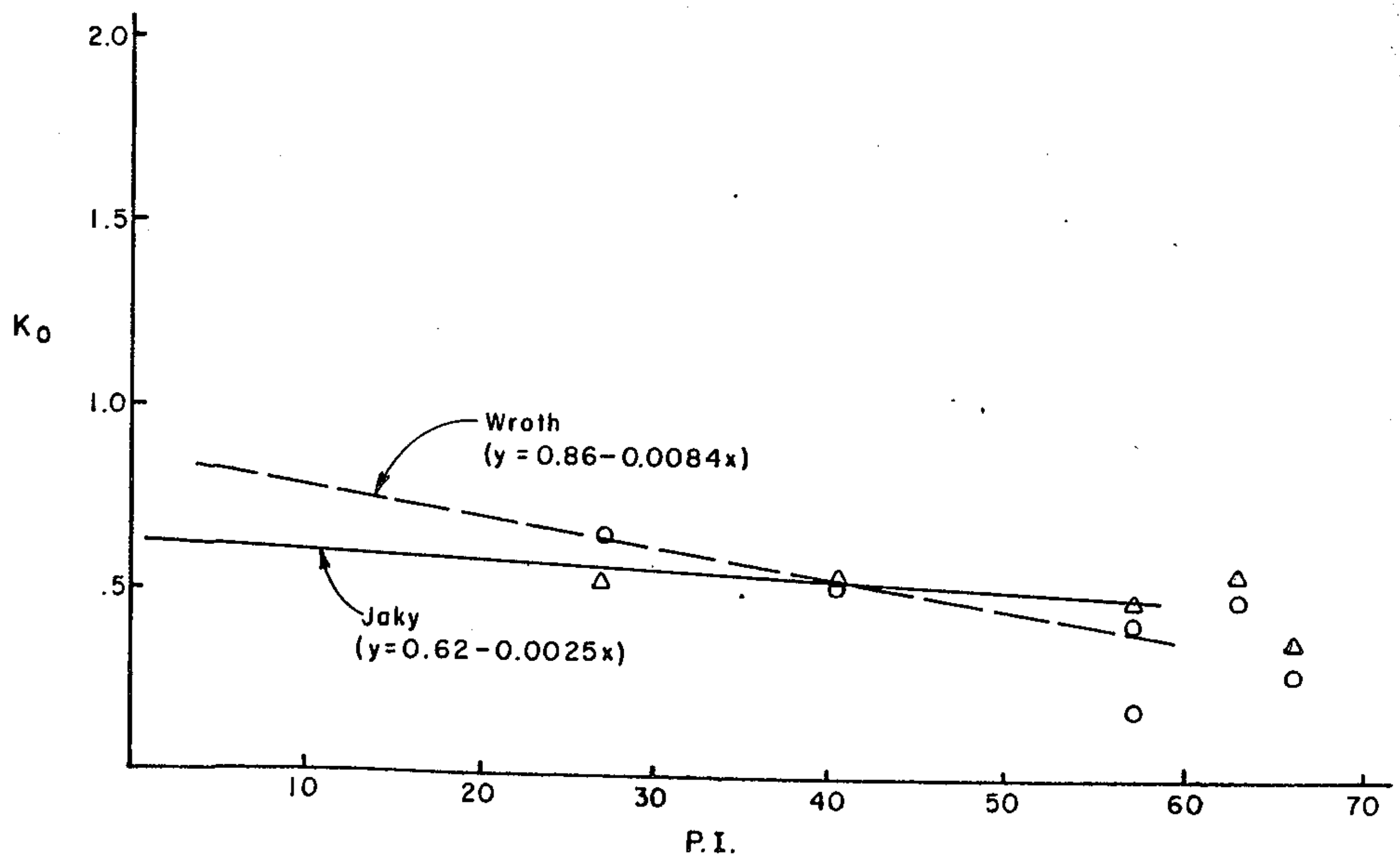


Figure 87 K_o VS PI, SITE 3

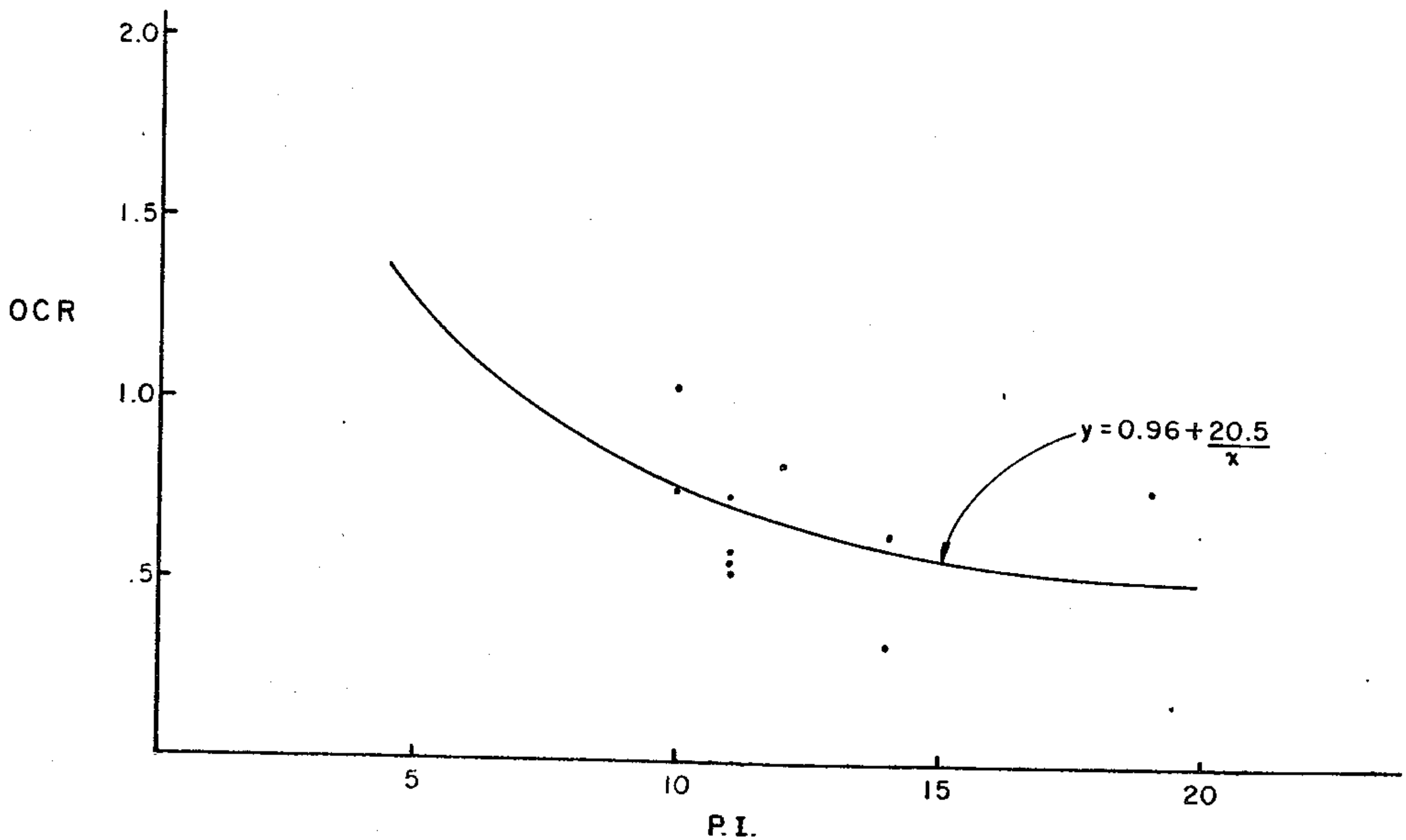


Figure 88 OCR VS PI, SITE 1

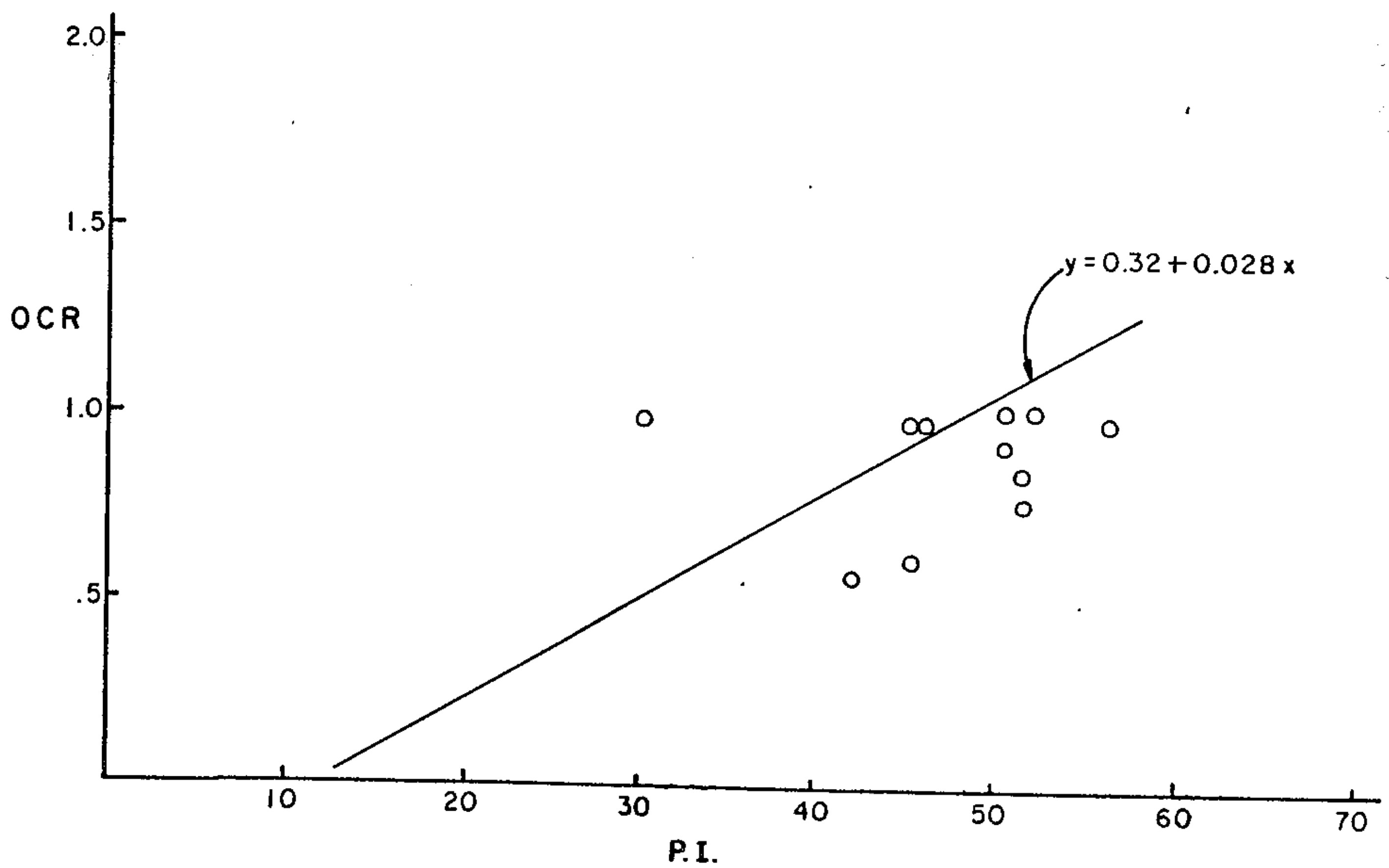


Figure 89 OCR VS PI, SITE 2

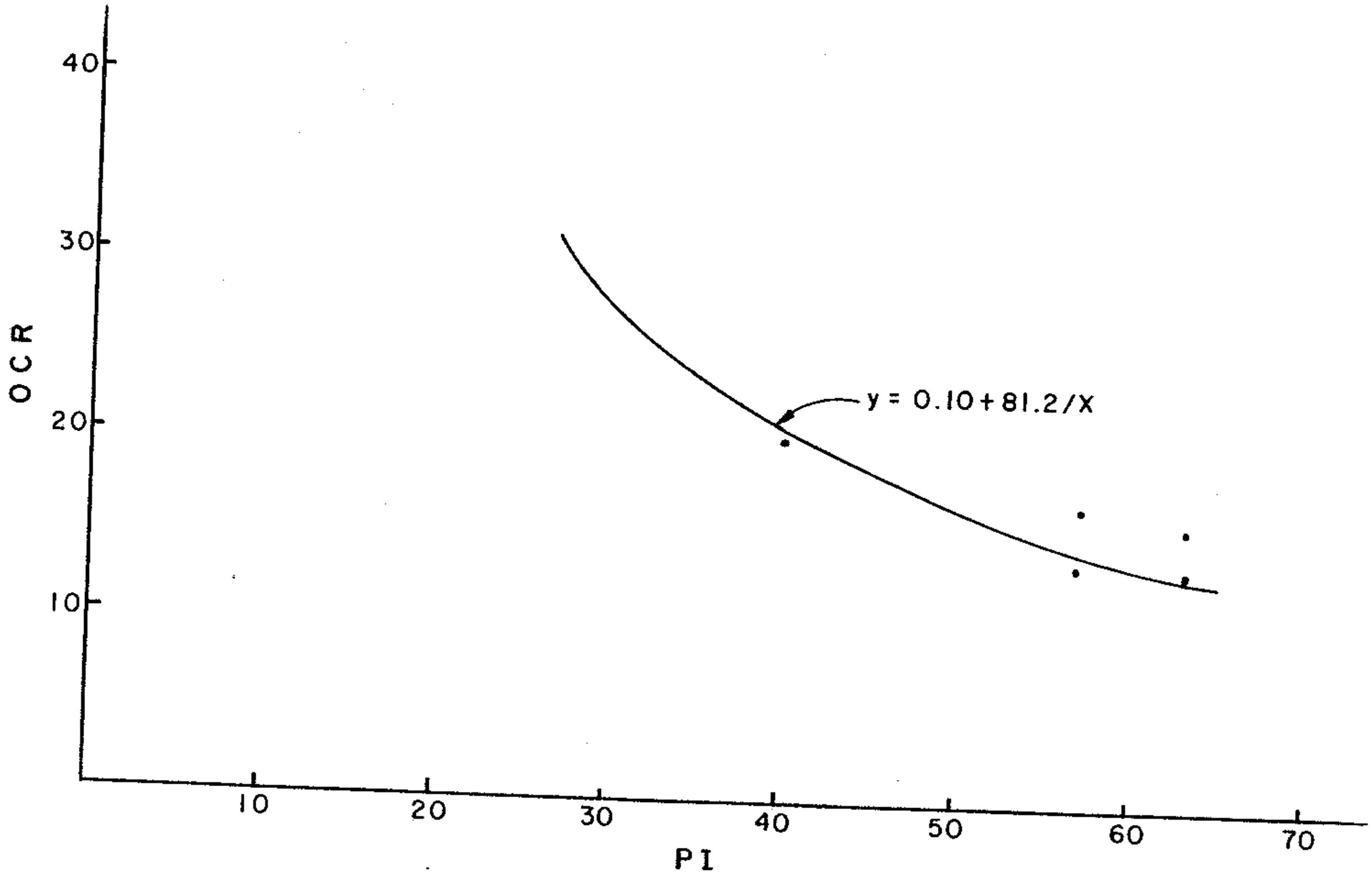


Figure 90 OCR VS PI, SITE 3

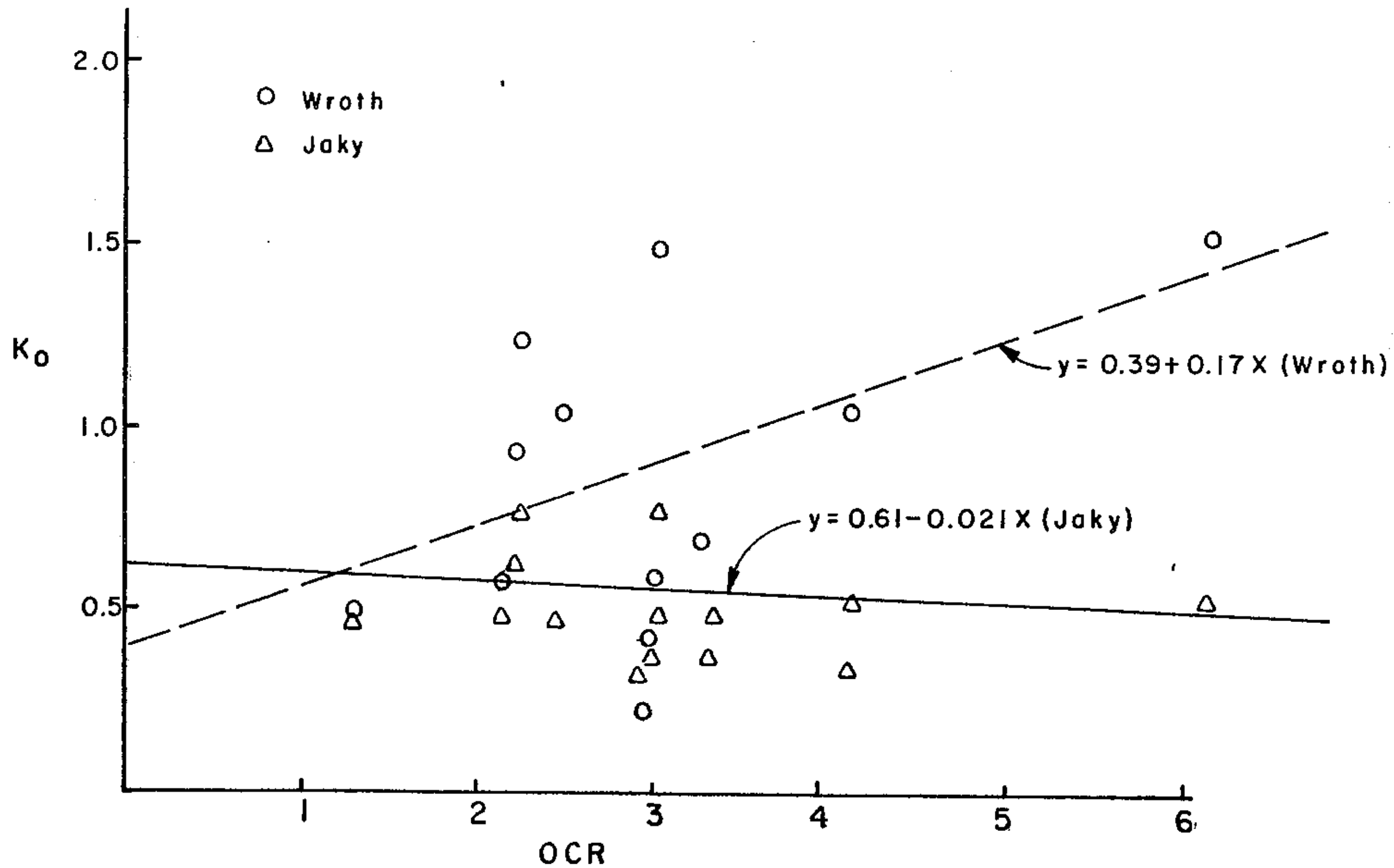


Figure 91 K_o VS OCR, SITE 1

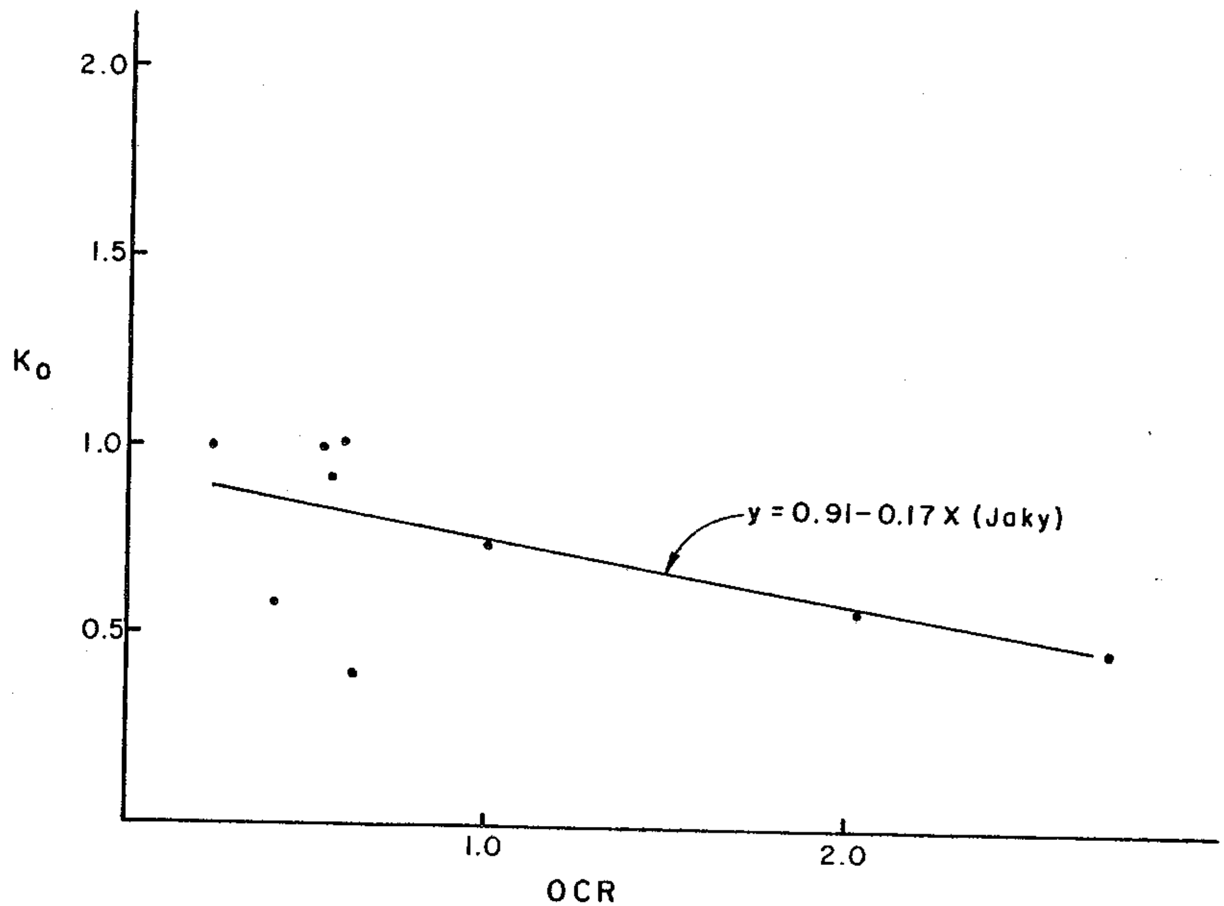


Figure 92 K_o VS OCR, SITE 2

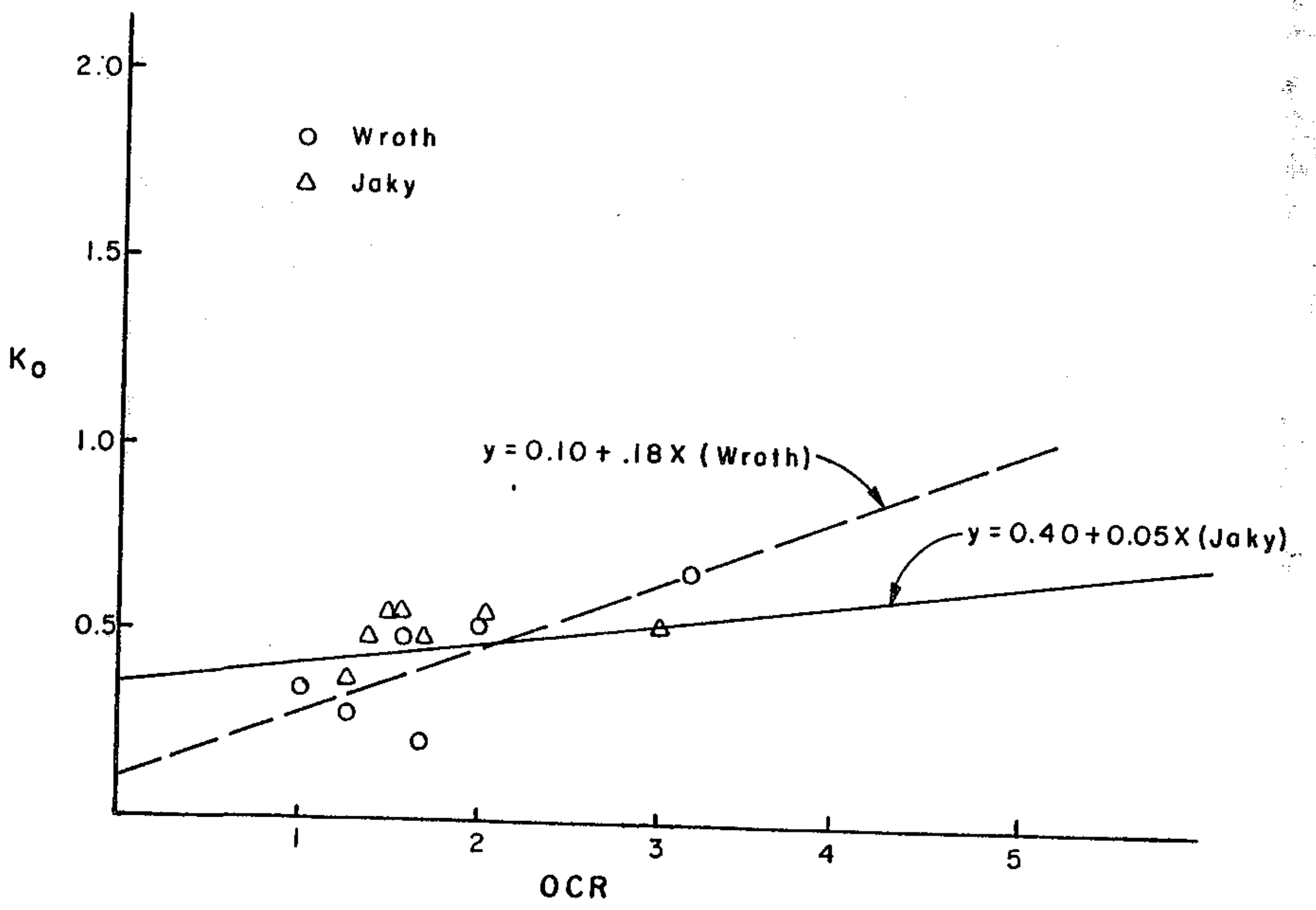


Figure 93 K_o VS OCR, SITE 3

CHAPTER 10. SOIL DISTURBANCE

Soil disturbance due to sampling, transportation, handling, and trimming has been of concern to geotechnical engineers for many years. It is known to significantly affect the results of soil strength and consolidation tests. As part of this project, it was proposed to examine one aspect of the problem, namely: sample size. It is generally accepted that the smaller the sample the greater the effect of soil disturbance.

It is a general practice outside California to recover 3-inch diameter undisturbed samples for laboratory testing; whereas, in California, 2-inch diameter samples are customarily obtained for this purpose. The intent of this phase of the study was to quantify the effect of this size variation for the soils encountered at the three test sites.

Three-inch samples were taken at all three sites using Shelby tubes in addition to the standard 2-inch samples recovered with the "California Sampler". The samples subject to test were divided into the following groupings: (1) three-inch samples, tested after extrusion; (2) three-inch samples, trimmed to two inches in diameter and tested; (3) two-inch samples tested after extrusion.

The laboratory tests that were conducted are as follows: (1) unconfined compressive strength; (2) unconsolidated undrained triaxial shear strength; (3) consolidated undrained triaxial shear strength with pore pressure measurements; and (4) consolidation tests.

Undrained Shear Strength Study

For Site 1, the undrained shear strength values from 2-inch samples are the highest (Figures 94 and 95). In Site 2, the unconfined compressive strengths of 3-inch samples were the highest. In this case the trimmed samples and 2-inch samples gave almost identical values (see Figure 96). This was not true in the case of CUe and UU tests (Figure 97). Here, 2-inch sample values were the highest; whereas, trimmed samples and 3-inch samples gave similar values. At Site 3, most of the soil layers consist of peat (Figure 98) and the values of unconfined compressed strength given by all three types of samples seem to be in the same general range. But the undrained shear strength values (CUe) (Figure 99) for 2-inch samples seem to be the highest.

Over Consolidation Ratio Study

For all three sites, the effect of trimming 3-inch samples to 2-inch samples seems to increase the O.C.R. values (Figures 100 through 102). In other words, the stress history is obscured. At Site 1, (Figure 100) the O.C.R. values from 2-inch samples were higher than from 3-inch samples; whereas, at Sites 2 and 3, the two sets of values were very close to each other.

Comparison With Parameters From In Situ Devices

The undrained shear strength values for Dutch Cone appear to be lower than those for other in situ devices tested at the three sites. The undrained shear strength values from both the French and Cambridge self boring pressuremeters generally seem to give the higher values. The other devices, for example: Iowa and Vane, develop values in between. In fact, no clearly discernible pattern emerges when the laboratory shear strength values are compared with field in situ values. A very general conclusion would be that the laboratory values are generally less (and in some cases much less) than the pressuremeter tests. These results are plotted in Figures 103 through 108.

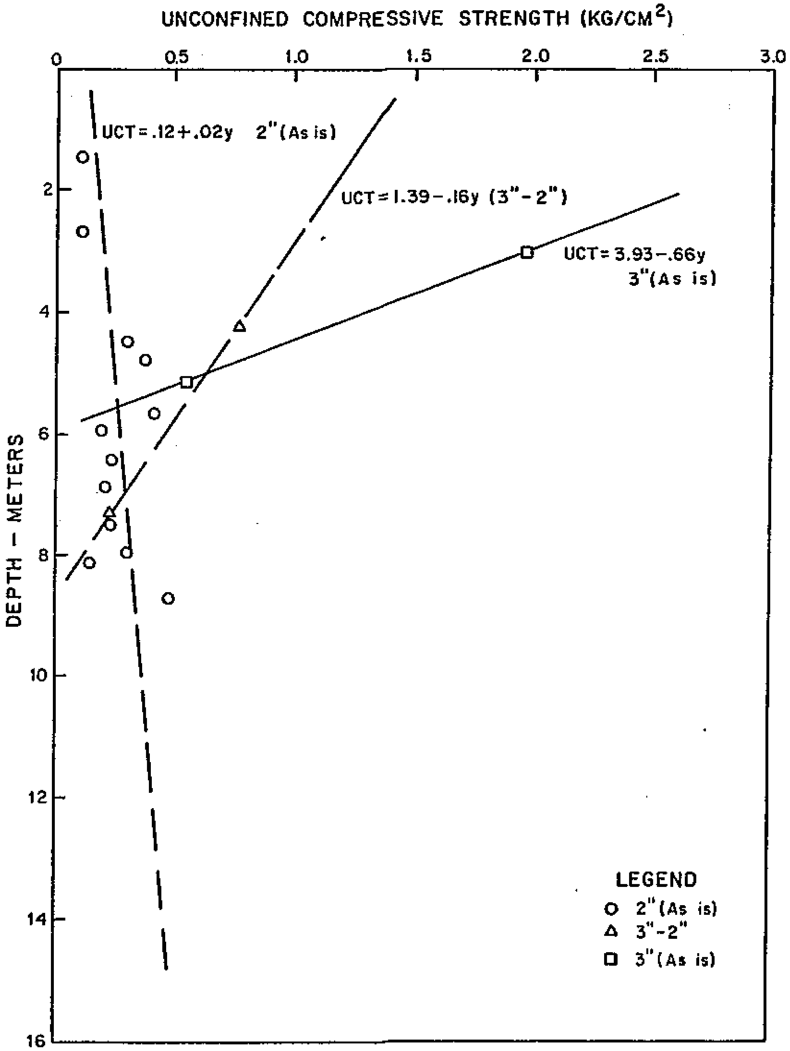


Figure 94 UNCONFINED COMPRESSIVE STRENGTH STUDY, SITE 1

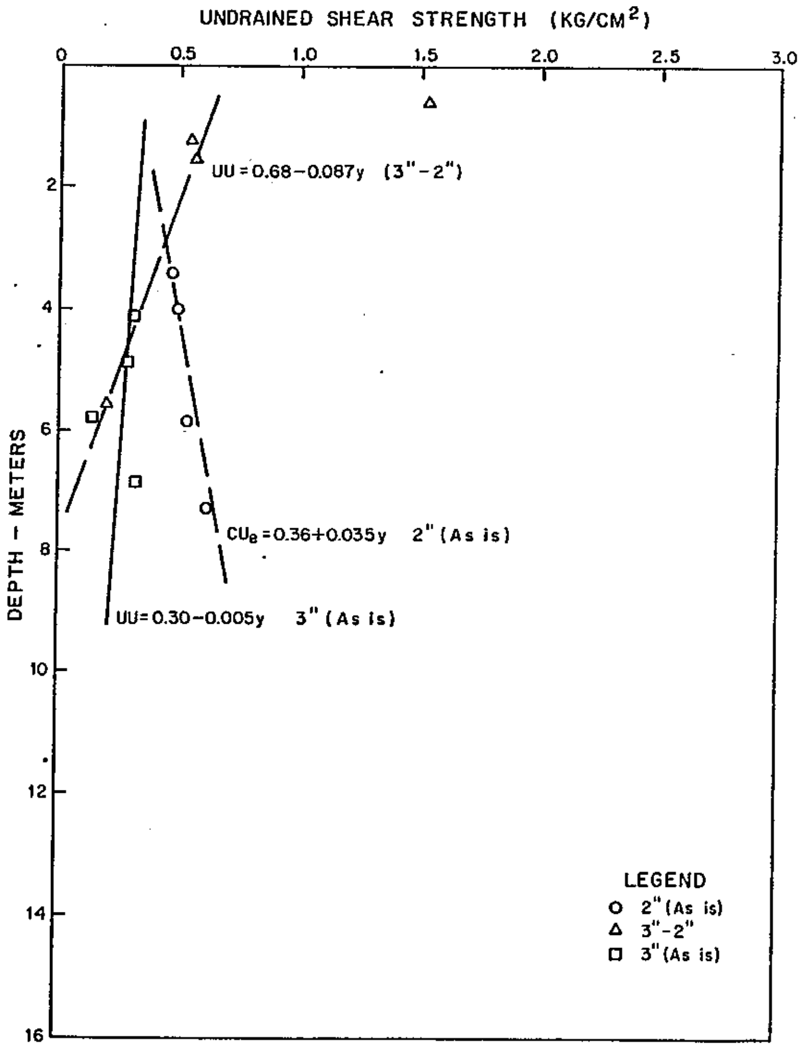


Figure 95 UNDRAINED SHEAR STRENGTH STUDY, SITE 1

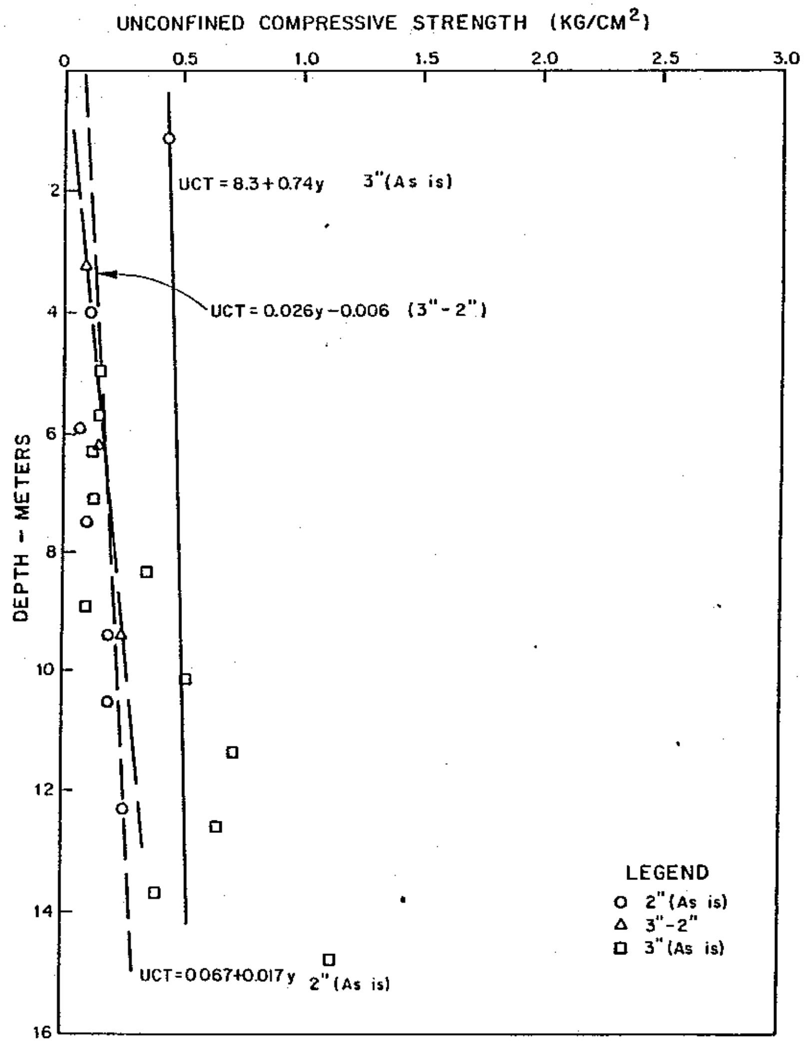


Figure 96 UNCONFINED COMPRESSIVE STRENGTH STUDY, SITE 2

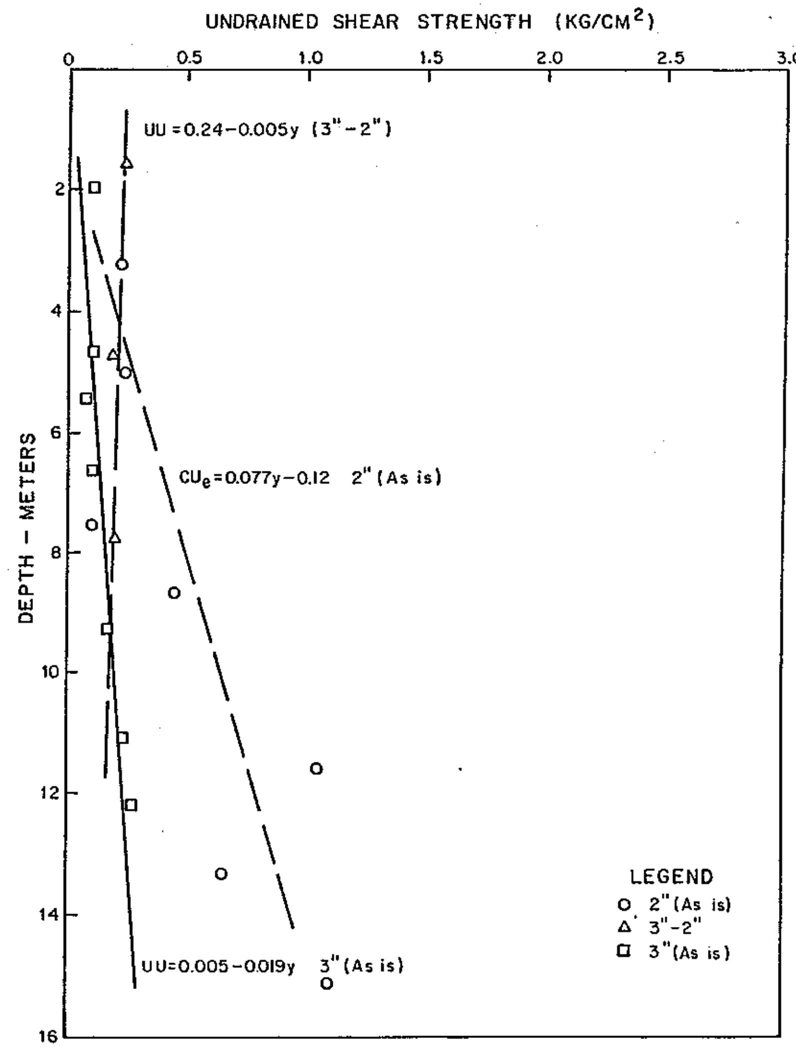


Figure 97 UNDRAINED SHEAR STRENGTH STUDY, SITE 2

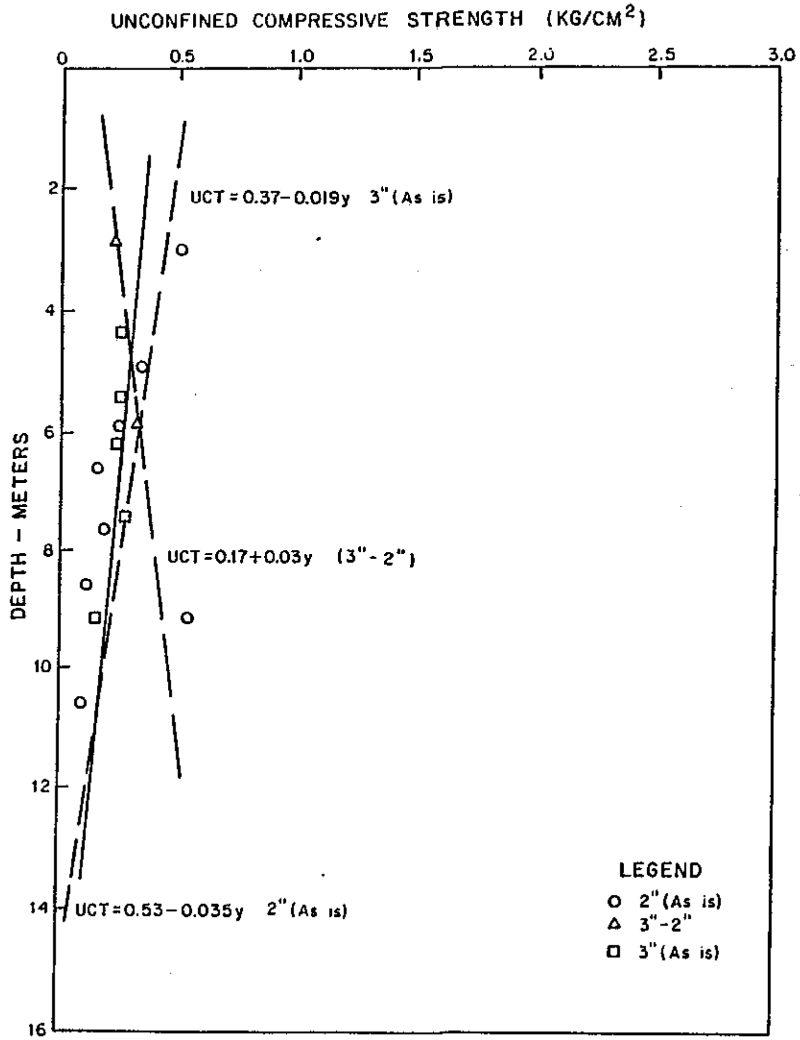


Figure 98 UNCONFINED COMPRESSIVE STRENGTH STUDY, SITE 3

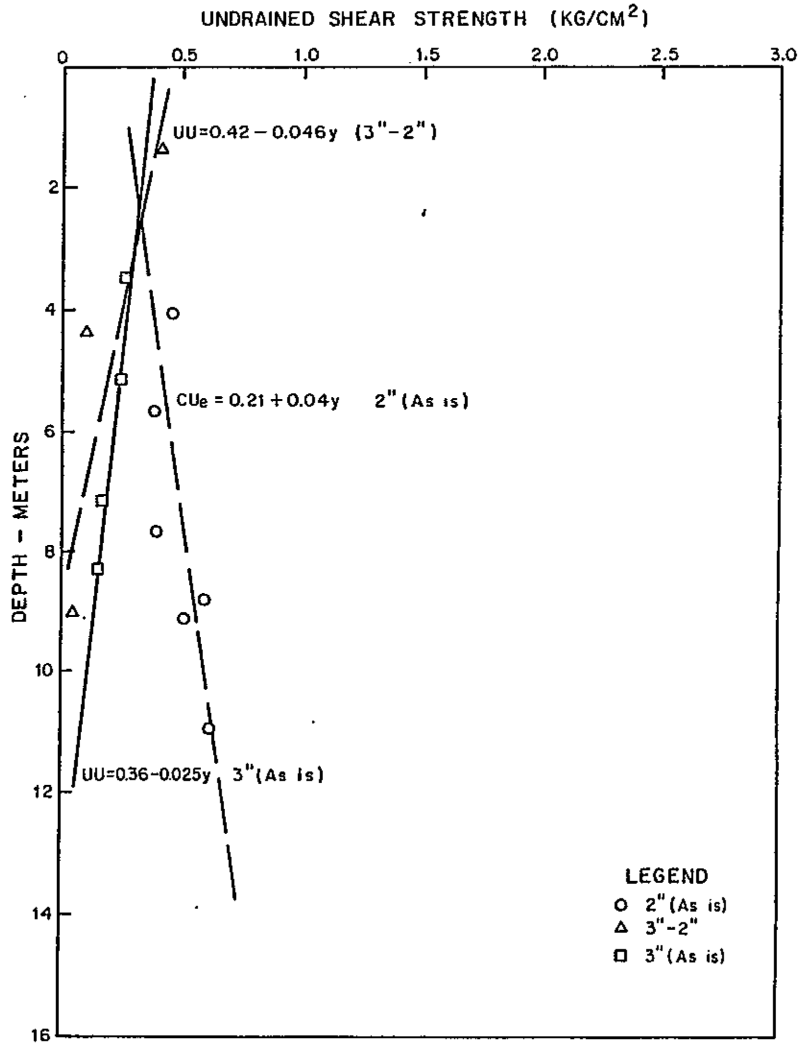


Figure 99 UNDRAINED SHEAR STRENGTH STUDY, SITE 3

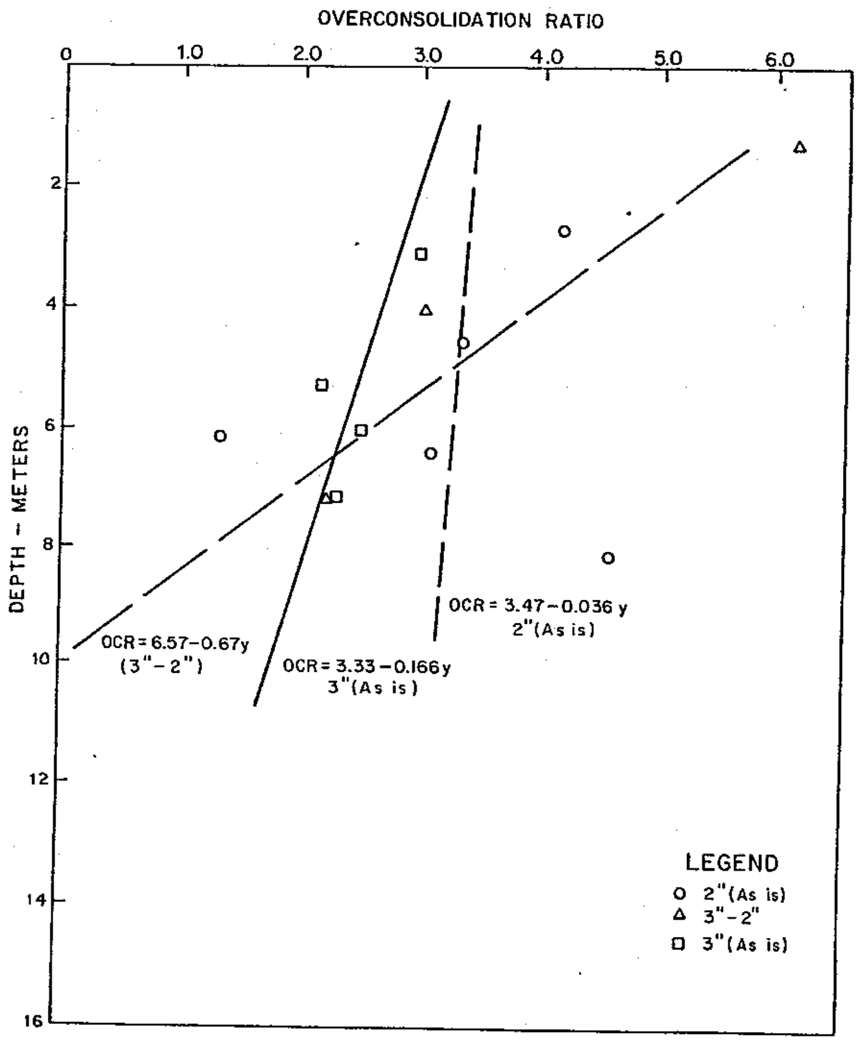


Figure 100 OVER CONSOLIDATION RATIO STUDY, SITE 1

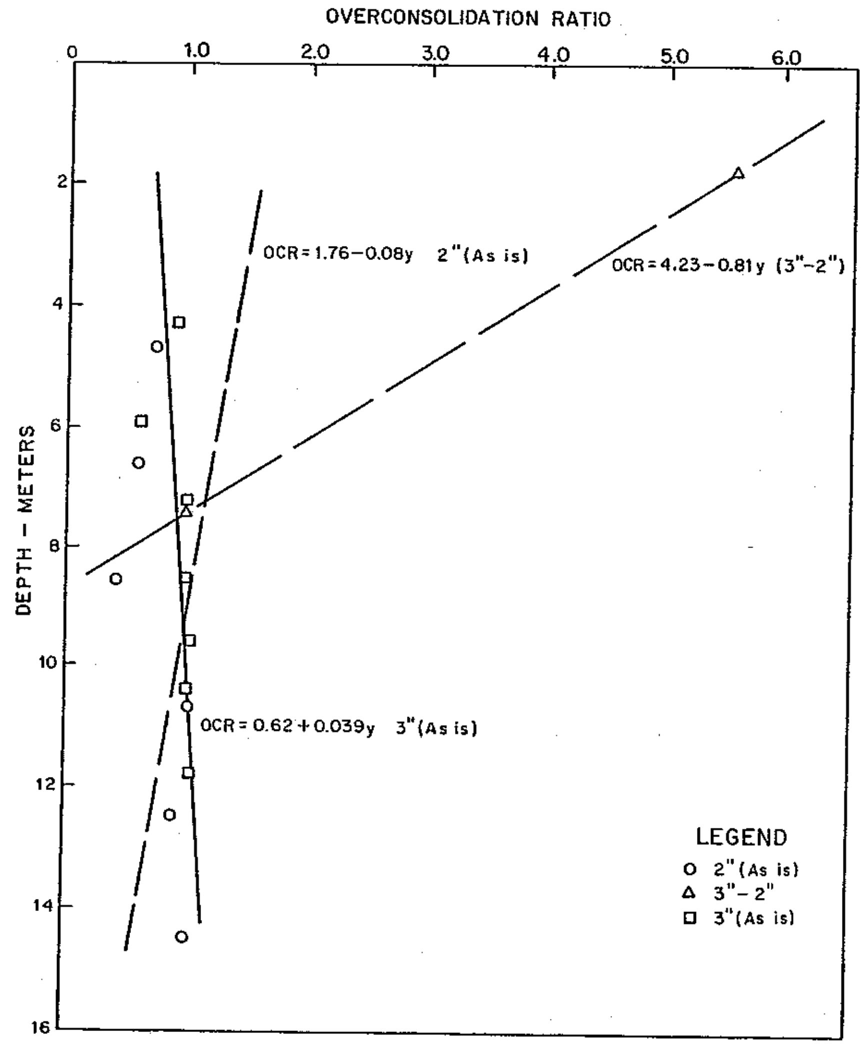


Figure 101 OVER CONSOLIDATION RATIO STUDY, SITE 2

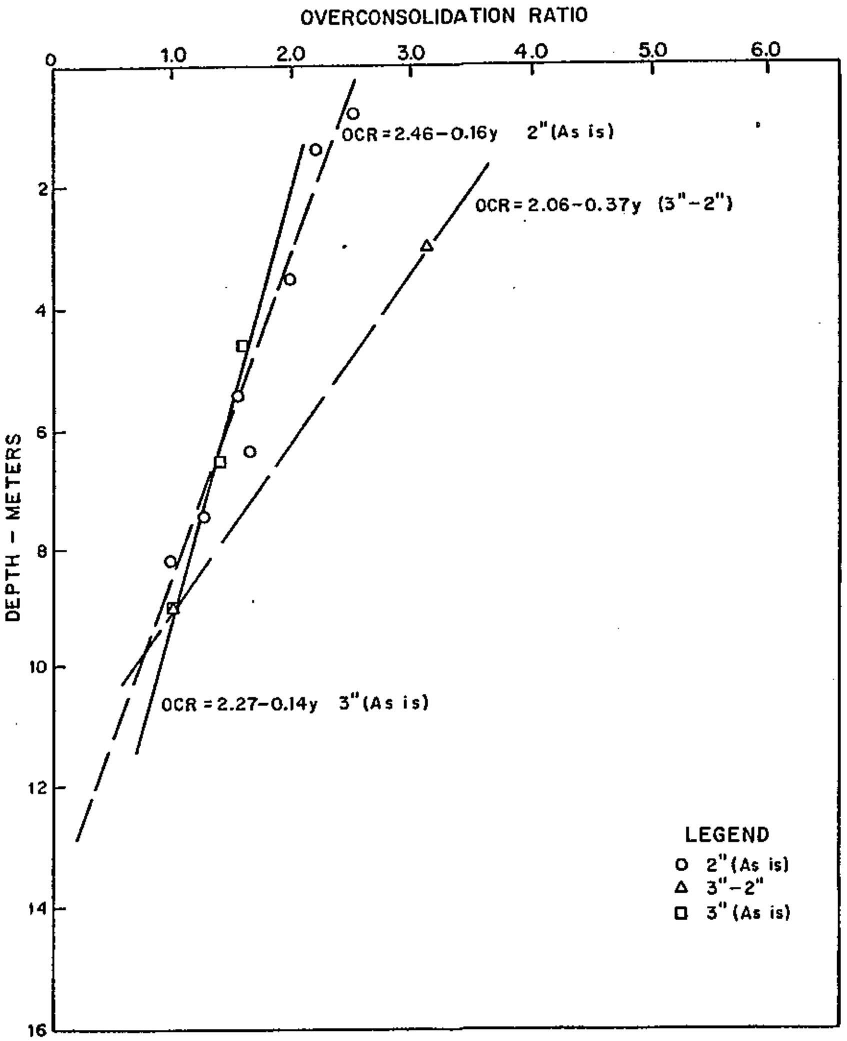


Figure 102 OVER CONSOLIDATION RATIO STUDY, SITE 3

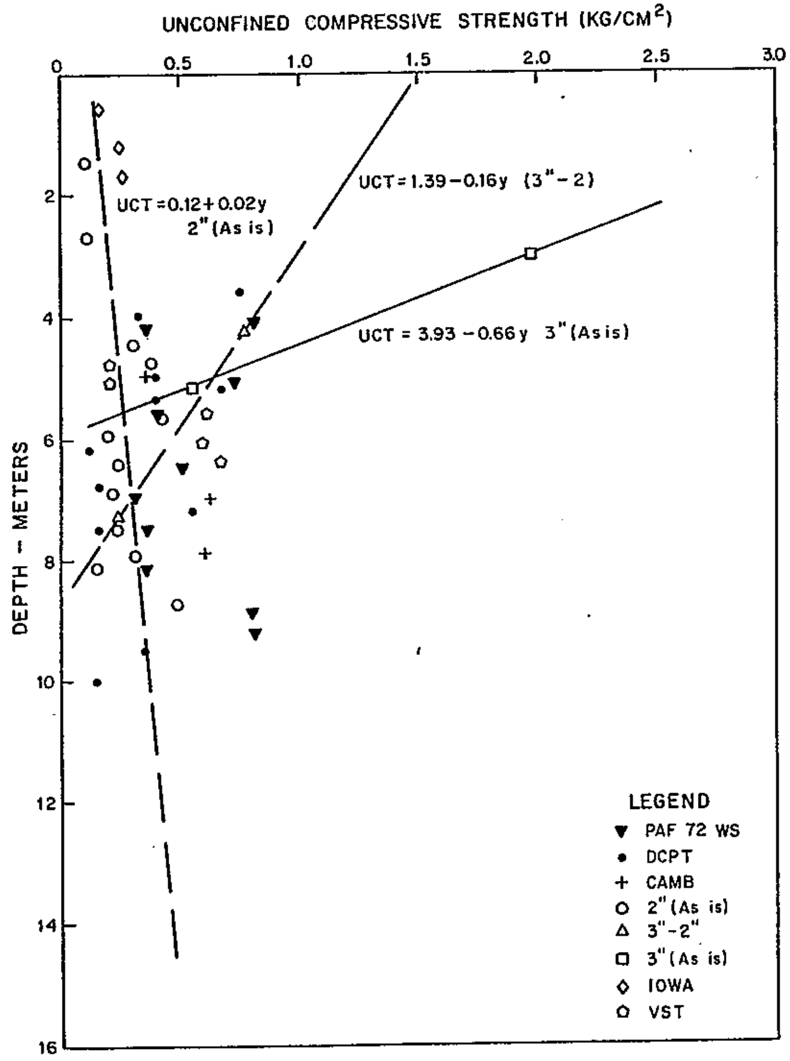


Figure 103 COMPARISON WITH PARAMETERS FROM IN SITU DEVICES, SITE 1

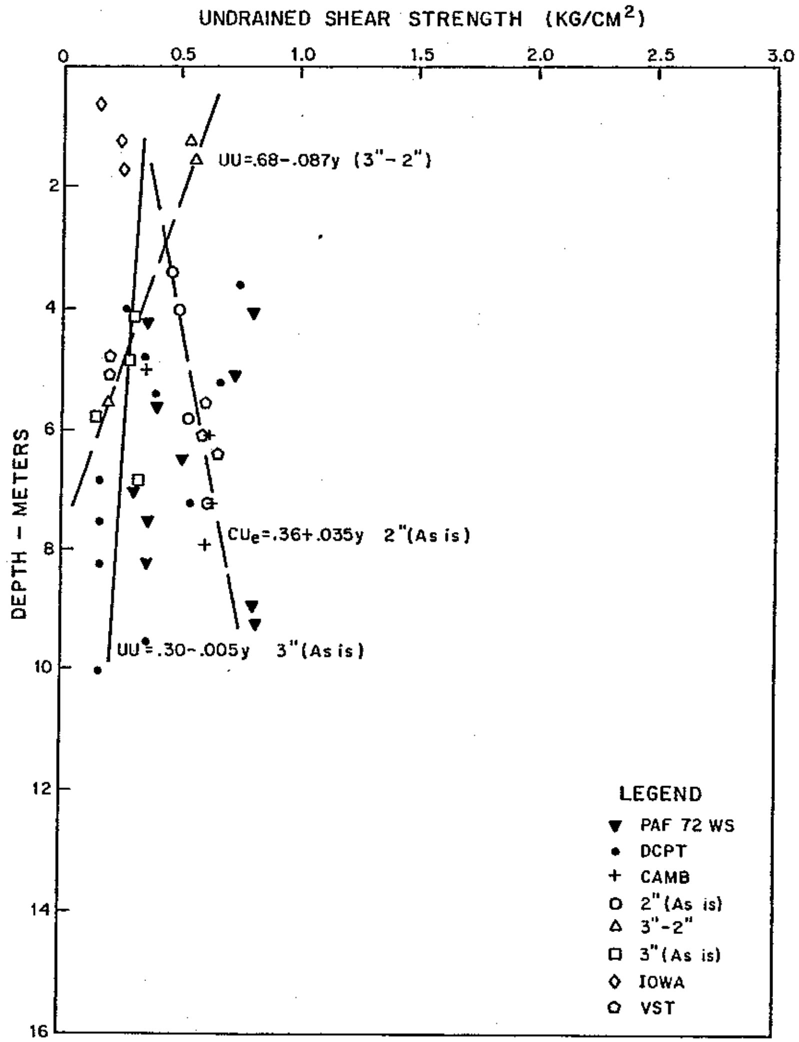


Figure 104 COMPARISON WITH PARAMETERS FROM IN SITU DEVICES, SITE 1

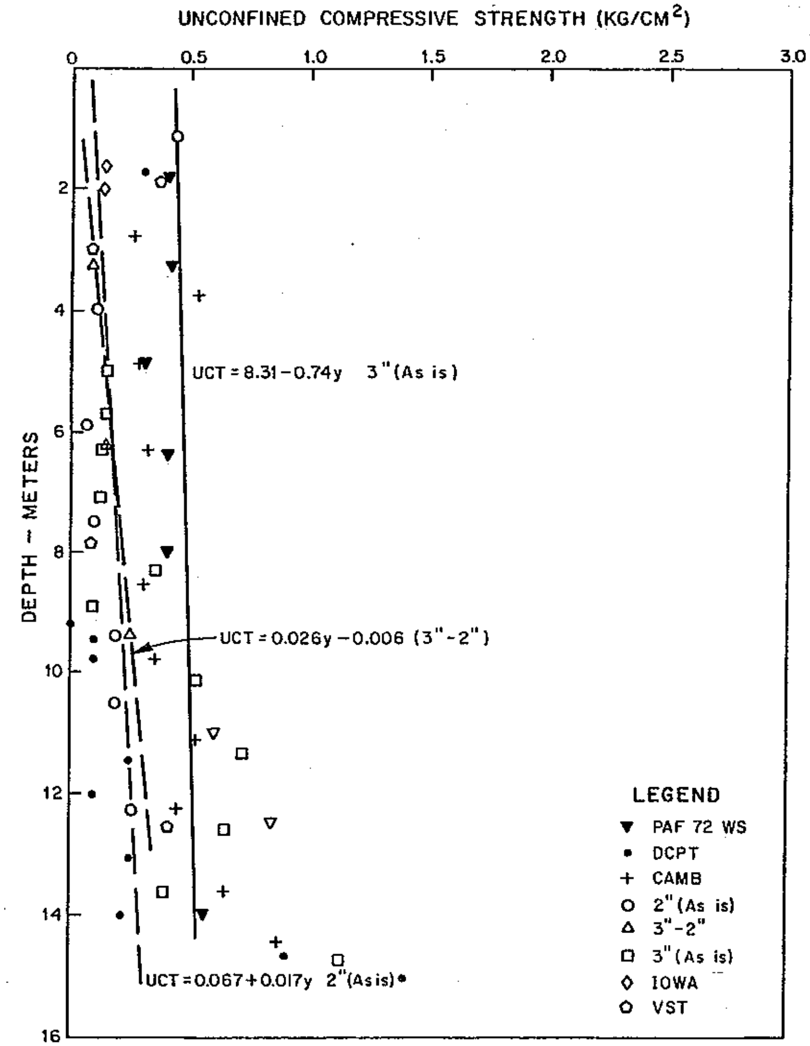


Figure 105 COMPARISON WITH PARAMETERS FROM IN SITU DEVICES, SITE 2

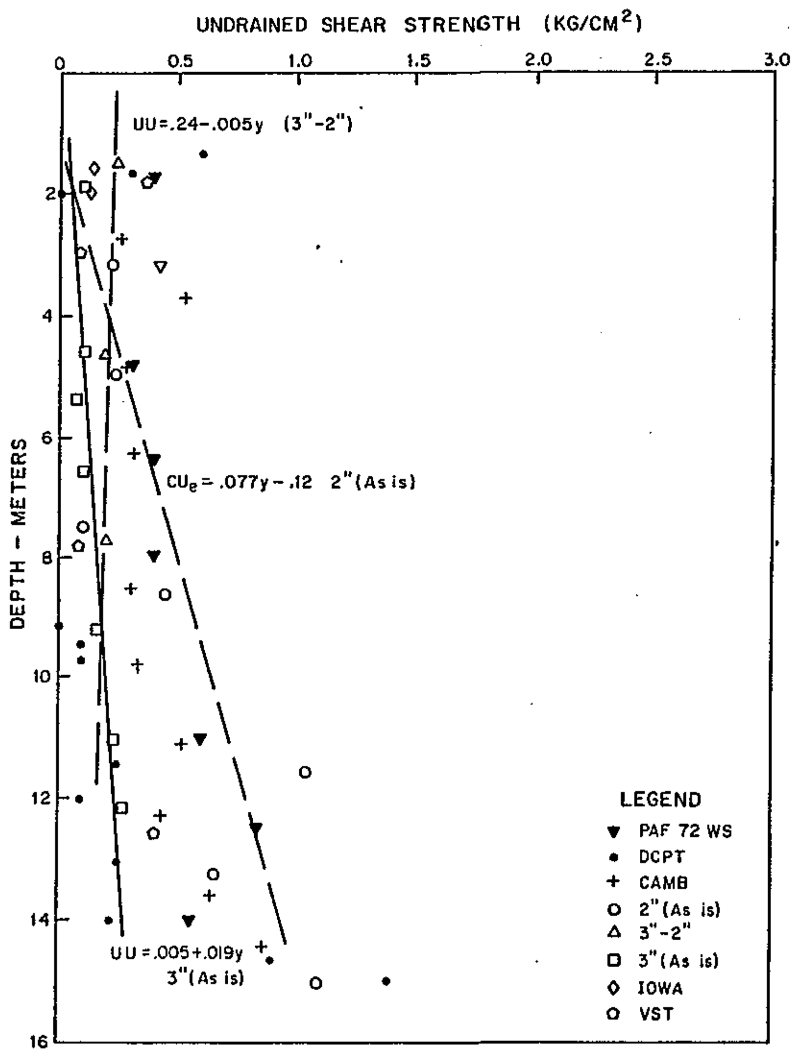


Figure 106 COMPARISON WITH PARAMETERS FROM IN SITU DEVICES, SITE 2

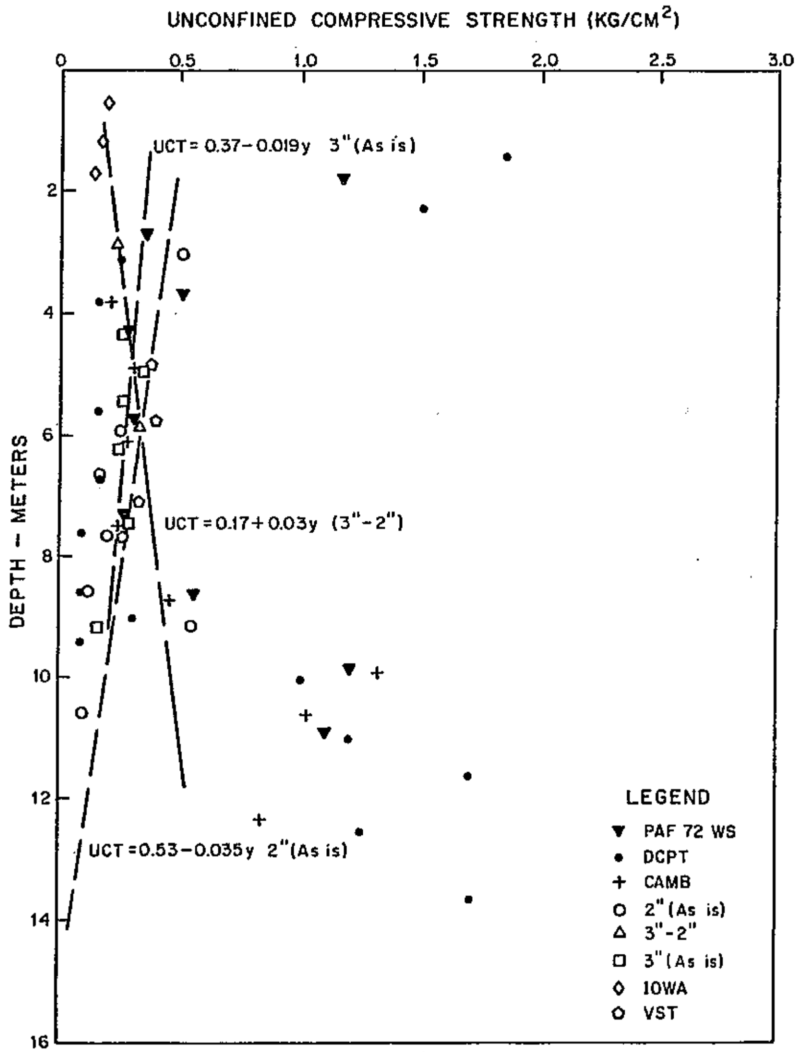


Figure 107 COMPARISON WITH PARAMETERS FROM IN SITU DEVICES, SITE 3

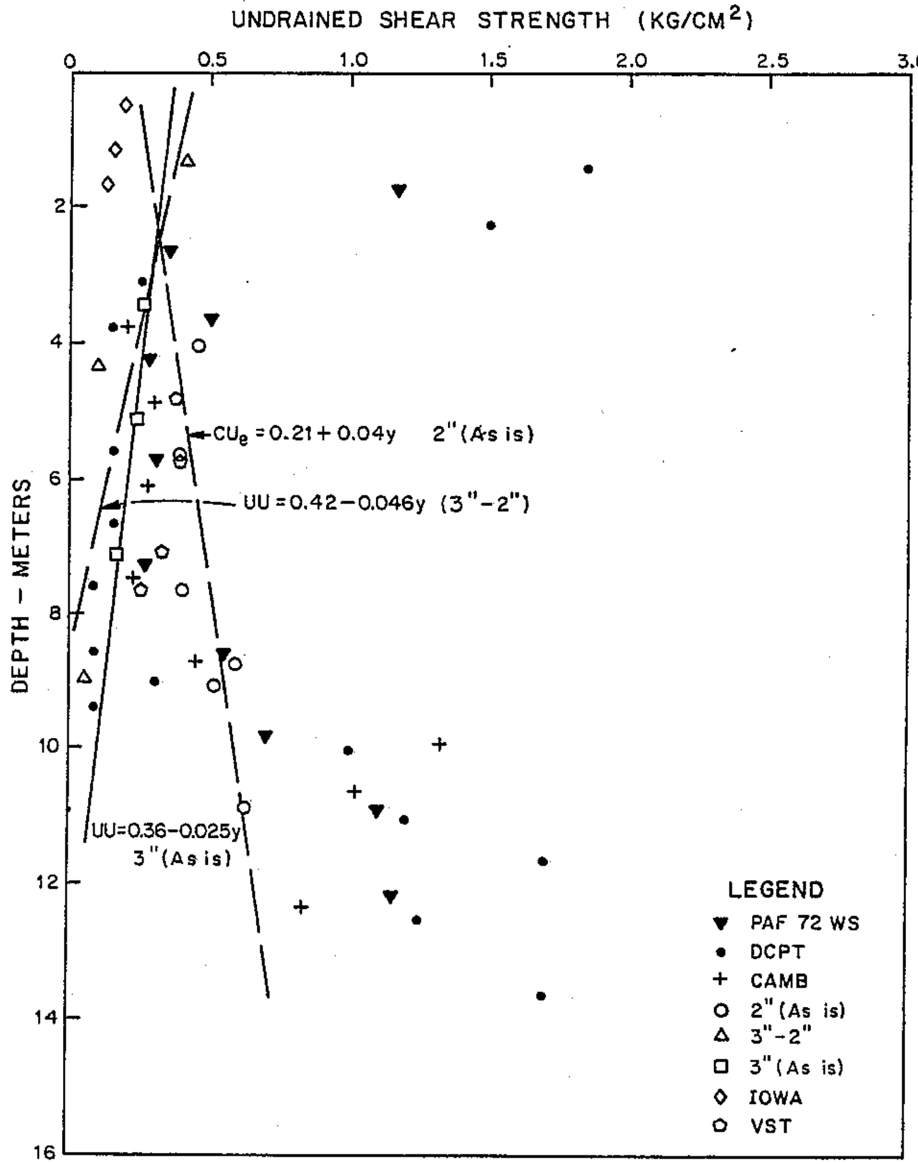


Figure 108 COMPARISON WITH PARAMETERS FROM IN SITU DEVICES, SITE 3

CHAPTER 11. CONCLUSIONS AND RECOMMENDATIONS

Conclusions - Evaluation

Iowa BSD

This probe can be used if the borehole does not cave or squeeze. Caltrans did not have success conducting tests with this equipment below groundwater table. This probe might be useful for shallow foundation design. The shear strength parameters (c & ϕ) obtained using this probe should be correlated further with laboratory values. However, the rapidity with which the shear strength parameters can be determined can result in significant savings under conditions in which the probe can be used. The soil disturbance problem seems to be critical since the test is conducted in the "smear" zone. Hence, great care should be exercised in the preparation of the borehole.

Vane S.D.

The undrained shear strength values for soft clays can be determined economically with reduced soil-structure disturbance. Even though Schmertmann (1975) and others object to use of correction factors suggested by Bjerrum, it seems that a definite correlation does exist that is related to plasticity index for most clays. The tests conducted with this probe were found to be quite reproducible. The test is relatively simple to perform and thus offers economic advantages.

Dutch Cone Penetrometer

The Dutch Cone penetrometer shows great promise for immediate application. It has the potential to provide an accurately controlled measurement of in situ ground conditions. The soil disturbance due to insertion seems to be minor in comparison to that resulting from conventional drilling and sampling operations. The test can be conducted quickly, and hence, economically. The depth of penetration is limited by the reaction that can be generated by the auxiliary equipment.

Standard Penetrometer

This probe, which has been in use for many years, is simple to operate. It lacks controls and needs further standardization of its various accessories. Many correlations involving standard penetrometer data exist in various parts of the world. Only regional correlations will be of any value. Even though it is likely to be superseded by some form of cone penetrometer in the long term, it will continue to be used by the industry. Research is in progress at present to gain a better understanding of this test.

Cambridge Self Boring Pressuremeter

This probe is a very sophisticated piece of in situ equipment. The system requires much electronic equipment to monitor and conduct tests. It develops test curves which yield much more information than any other in situ test. The theory of interpretation is well understood now. It is particularly useful for large projects where finite element analysis is absolutely essential and in which the soils are homogeneous and free of shells or sharp objects. Test hole preparation prior to the self boring process through the overburden requires much care. The small drill rig to which this probe was adapted was found to have insufficient power to retract the probe from the hole in some cases.

Test Results

Iowa B.S.D.

The field results from the Iowa Borehole Shear Device were from shallow depths only, and hence could not be fully compared with other in situ devices.

Vane S.D.

The vane shear strength values were found to be between undisturbed and remolded values of unconfined compressive strength.

Dutch Cone Penetrometer

The Dutch Cone test data seems to be the most promising of all the five in situ probes evaluated. The data obtained enables not only reconstruction of the soil profile on a given project, but also estimation of the undrained shear strength values at any depth. The test data can also be used for pile design and analysis.

Standard Penetrometer

The standard penetrometer test data could be used in its traditional role or could be correlated with Dutch Cone data.

Cambridge Self Boring Pressuremeter

Although insufficient test data were generated on this project for definitive correlations, there appears to be a well defined relationship between the Cambridge self boring pressuremeter and Dutch Cone data. The unique pressuremeter theory permits derivation of more soil parameters from one in situ test than any of the other in situ devices. As this probe has yet to gain wide acceptance, much additional correlative work with other in situ devices is needed.

Soil Disturbance

The data indicate that shear strength values from 3-inch samples are generally higher than those from 2-inch samples. This implies that values from 2-inch samples used in design lead to conservative geotechnical conclusions. Also, in general, the values from 3-inch samples are closer in magnitude to those from the Cambridge and French pressuremeters. Trimming 3-inch samples to 2-inch samples results in shear strength values between those from 2-inch samples and 3-inch samples, with those from 2-inch samples being generally lower.

Recommendations - Probes

Iowa BSD

1. It is recommended that shear plates of larger area be used in future commercial models.

2. The working table and the buzzer should form part of the accessory equipment.

Vane S.D.

1. ASTM specifications for conducting vane shear tests should specify methods for calibration on the various accessories, such as the proving ring. In addition, rod lengths should be standardized.

2. This probe is recommended for use in all geotechnical investigations in soft to medium stiff clays and soils.

Dutch Cone Penetrometer

1. Regional correlation between Dutch Cone penetrometer and various geotechnical parameters should be developed.

2. The two charts (Schmertmann and Begemann) used by the industry to reconstruct soil profiles were found to require further development for application to California soils.

Standard Penetrometer

1. Standardization of the various accessories of this probe (on the drill rig) is needed.

2. For any particular region it is recommended that correlations between this test (SPT) and the Dutch Cone penetrometer test be established for the various types of soils encountered.

Cambridge Self Boring Pressuremeter

1. This probe, while promising, is a relatively complex device. A great deal of additional experience with it is required prior to general acceptance.

2. It should be adapted to a large drill rig such as the Mobil B-61.

Further Study

1. Based on this research effort and the work on the French probes(1,2) a correlation has been found to exist between Dutch Cone Penetrometer test data and self boring pressuremeter test data. Further research in the development of this correlation could prove beneficial.

2. Dutch Cone test data have been found useful in identifying soil types. The two charts in use (Begemann's and Schmertmann's) were found to be inadequate for this purpose. These two charts could be extended or a new chart developed to reconstruct soil profiles.

IMPLEMENTATION

Copies of this final report will be distributed to the various Caltrans Headquarters Offices and 11 Transportation Districts, and to the Federal Highway Administration for distribution within their organization.

The in situ probes evaluated in this study will be used in routine geotechnical investigations by the Transportation Laboratory, initially. Training courses will be set up later for practical use by the Districts. Standard test methods are already under preparation for the Cambridge Probe and Dutch Cone.

As an indication as to which system may be appropriate for a given project, Table 5 has been developed.

TABLE 5. SUGGESTED GUIDE FOR SELECTION OF A PROBE

	NAME OF PROBE	ORIGINALLY DEVELOPED BY	PROBE EQUIPMENT DESCRIPTION	PARAMETERS OBTAINED	APPLICATION	LIMITATIONS	ADVANTAGES
1.	IOWA BORE-HOLE SHEAR DEVICE	HANDY & FOX, USA (1968)	BORE-HOLE TEST APPARATUS LOWERED IN BORE-HOLE AND TWO GROOVED PLATES ARE EXPANDED TO CONTACT WALLS OF BORING. EXPANSION FORCE AND PULLING FORCE ARE MONITORED TO OBTAIN NORMAL STRESS AND SHEAR STRESS. MOHR-COULOMB TOTAL STRESS FAILURE ENVELOPE IS GENERATED BY RELATIONSHIP OF CONTACT PLATE AREAS TO APPLIED FORCE.	VALUES OF ϕ AND c FOR SAND, SILT AND CLAY ARE REALISTIC WHEN COMPARED TO LABORATORY TEST RESULTS. IN CLAYS, IT WOULD BE CLOSE TO UNDRAINED SHEAR STRENGTH. IN SANDS AND SILT, IT WOULD BE CLOSE TO DRAINED SHEAR STRENGTH.	CAN BE USED FOR SHALLOW FOUNDATION DESIGN IN SILTS, CLAYS AND SANDS.	IT NEEDS A PREDRILLED HOLE. IF THERE IS CAVING, CANNOT BE USED.	CAN BE CARRIED FOR SLOPE STABILITY INVESTIGATIONS, RAPIDITY OF TESTING.
2.	VANE SHEAR DEVICE	LYMAN CADLING, SWEDEN (1947)	VANES INSERTED INTO SOFT, FINE-GRAINED SOILS TORQUE APPLIED TO A CONNECTING ROD WHICH EXTENDS TO THE SURFACE.	UNDRAINED SHEAR STRENGTH (HORIZONTAL), SENSITIVITY IN CLAYS.	CAN BE USED IN SOFT CLAYEY OR SILTY SOILS - STABILITY ANALYSIS, SHALLOW FOUNDATION DESIGN, BEARING CAPACITY.	LIMITED TO SOFT FINE-GRAINED COHESIVE SOILS FREE OF ROCK, SHELLS, ETC. CANNOT BE USED IN STIFF SOILS.	TEST CAN BE EASILY PUSHED IN SOFT CLAYS, PORTABLE, SIMPLE AND ECONOMICAL TO OPERATE, TEST RESULTS REPEATABLE.
3.	DUTCH CONE PENETROMETER	PUBLIC WORKS, THE NETHERLANDS (1935) GONDSE MACHINE-FABRIEK, THE NETHERLANDS (1959)	CONE WITH OR WITHOUT SLEEVE PUSHED INTO THE SOIL AND RESISTANCE AND FRICTION MEASURED.	UNDRAINED SHEAR STRENGTH.	DETERMINATION OF SOIL TYPE, PILE DESIGN, BEARING CAPACITY OF BOTH SHALLOW AND DEEP FOUNDATIONS.	DEPTH OF PENETRATION LIMITED BY THRUST CAPABILITY OF DRILL RIG.	THE TEST IS FAST, SIMPLE, AND ECONOMICAL - CAN BE USED IN ALL SOILS - USING CORRELATIONS AND SEMI-EMPIRICAL METHODS HAS GREAT POTENTIAL FOR MANY VARIED FOUNDATION CONDITIONS.
4.	STANDARD PENETROMETER	H. A. MOHR GOW COMPANY SPRAGUE & HENWOOD, USA (1927-30)	A 140 LB. HAMMER WITH A DROP OF 30 INCHES DRIVES A SAMPLER.	RESISTANCE IN TERMS OF BLOW COUNTS CAN BE CONVERTED TO ϕ, RELATIVE DENSITY.	FRICTION ANGLE OF SANDY SOILS - BEARING CAPACITY OF SOILS FOR SHALLOW FOUNDATIONS, LIQUEFACTION STUDIES OF SAND DEPOSITS.	POOR REPRODUCIBILITY, AND WIDE VARIABILITY OF TEST DATA.	RELATIVELY INEXPENSIVE, SIMPLE TO USE AND OPERATE, PRESENTLY WIDELY ACCEPTED.
5.	CAMBRIDGE SELF BORING PRESSUREMETER	C. PETER WROTH, JOHN M. O. HUGHES CAMBRIDGE UNIV., UK (1974)	CONSISTS OF A SELF-TUNNELLING MACHINE, AND A MEASUREMENT MODULE.	UNDRAINED SHEAR STRENGTH OF CLAYS, SILTS AND FINE SANDS. VARIOUS MODULI (E_o, E_s AND E_f) LIMIT PRESSURES (P_L AND P_f) STRAIN AT FAILURE.	CAN BE USED IN SOFT TO MEDIUM CLAYS, SILTS AND FINE SANDS - STABILITY ANALYSIS - FOUNDATION (SHALLOW & DEEP) DESIGN - PILE DESIGN (LATERAL REACTION) - PARAMETERS IDEALLY SUITED FOR DIRECT APPLICATION IN FINITE ELEMENT ANALYSIS.	WILL NOT PENETRATE GRAVEL, DRAINAGE EFFECTS CANNOT BE CONTROLLED, COMPLEX BY TODAYS STANDARDS.	TESTS PERFORMED IN VIRTUALLY UNDISTURBED SOIL - MANY PARAMETERS FROM ONE TEST.

ABBREVIATIONS AND SYMBOLS

Chapter 1

HPR = Highway Planning and Research

Chapter 2

DCP = Dutch Cone Penetrometer

CP = Cambridge Pressuremeter

ISD = Iowa Borehole Shear Device

VSD = Vane Shear Device

SP = Standard Penetrometer

Chapter 3

UU = Unconsolidated undrained triaxial compression shear test

CUe = Consolidated undrained triaxial compression shear test with pore pressure measurements

OCR = Over Consolidation Ratio

S_u = Undrained shear strength

P'_o = effective overburden pressure

P'_p = effective preconsolidation pressure

Chapter 4

BSD = Borehole Shear Device

psi = pounds per square inch

CO_2 = Carbon Dioxide

$\Delta\sigma n$ = Incremental value of normal stress

τ = pulling stress

σn = Normal stress

c = Cohesion

ϕ = friction angle

CBR = California Bearing Ratio

Chapter 5

T = Torque

D = Vane Diameter

H = Vane Height

S = Shear Strength

k = constant for a given vane size

p = gauge reading

Chapter 6 Dutch Cone Penetrometer

q_c = Cone Resistance or cone bearing capacity

Chapter 7 Standard Penetrometer

N = blow count

Chapter 8 Cambridge Self Boring Pressuremeter

CP = Cambridge Probe

CSB = Cutter Set Back

TM = Tunnelling Module

MM = Measurement Module

Chapter 9 Test Results and Discussion

UCT = unconfined compressive strength test
C_n = Correction factor for overburden pressure
N_{field} = Blow counts corrected for overburden pressure
K_o = Coefficient of Earth Pressure at rest
Cu = peak undrained shear strength
DCPT = Dutch Cone Penetrometer Test
VST = Vane Shear Test
HP = Hand Penetrometer
TV = Torvane
E_o = Initial Target Modulus
E_s = Subtangent modulus
E_f = Failure modulus
P_e = Practical Limit Pressure
P_L = Theoretical Limit Pressure
M&R = Marsland and Randolph approach
PI = Plasticity Index
OCR = Over Consolidation Ratio

Chapter 10 Soil Disturbance

LL = Liquid Limit
PL = Plastic Limit

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